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VOL. 72 DECEMBER, 1946 No. 10

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AND

DISCUSSIONS

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Report: Opportunities for Sanitary Engineers in the Postwar Period: Progress Report of the Committee on Advancement of Sanitary Engineering of the Sanitary Engineering Division.	May, 1946	
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Tommerup, Carl C. H. Rigid-Frame Structures Subject to Nonuniform Thermal Action. Discussion in Nov., 1946.	June, 1946	Jan. 1, 1947
Middlebrooks, T. A., and Jervis, William H. Relief Wells for Dams and Levees. Discussion in Oct., Dec., 1946.	June, 1946	Jan. 1, 1947
Report: Advances in Sewage Treatment and Present Status of the Art: Third Progress Report of the Committee of the Sanitary Engineering Division on Sewerage and Sewage Treatment.	June, 1946	
Stevens, J. W. Truck Speed and Time Loss on Grades. Discussion in Dec., 1946.	Sept., 1946	Feb. 1, 1947
Underwood, P. H. Space Resection Problems in Photogrammetry. Discussion in Dec., 1946.	Sept., 1946	Feb. 1, 1947
McGuinness, Charles L. Recharge and Depletion of Ground-Water Supplies.	Sept., 1946	Feb. 1, 1947
de Vries, Karl. Strength of Beams as Determined by Lateral Buckling. Discussion in Dec., 1946.	Sept., 1946	Feb. 1, 1947
Ward, C. N., and Hunt, Henry J. Correction of Tailwater Erosion at Prairie Du Sac Dam.	Oct., 1946	Mar. 1, 1947
Benscoter, Stanley U. Matrix Analysis of Continuous Beams	Oct., 1946	Mar. 1, 1947
Machis, Alfred. Experimental Observations on Grouting Sands and Gravels.	Nov., 1946	Apr. 1, 1947

NOTE.—The closing dates herein published are final except when names of prospective discussers are registered for special extension of time.

* Publication of closing discussion pending.

CONTENTS FOR DECEMBER, 1946

PAPERS

PAGE

Cleaning and Grouting of Limestone Foundations, Tennessee Valley Authority: A Symposium.
By Arthur L. Pauls, and T. F. Taylor 1329

REPORTS

Stewardship of Professionalism Given High Priority: Address at the Annual Convention, Spokane, Wash., July 17, 1946.
By W. W. Horner 1371

DISCUSSIONS

Strength of Thin Steel Compression Flanges.
By L. C. Maugh and L. M. Legatski 1381

Landslide Investigation and Correction.
By D. P. Barnes 1387

Thrust Exerted by Expanding Ice Sheet.
By Ernest Brown, B. R. McGrath, and Robert E. Kennedy 1391

Model Study of Brown Canyon Debris Barrier.
By Ettore Scimemi, and J. W. Johnson 1401

Relief Wells for Dams and Levees.
By W. A. Wall and C. A. Stone 1407

Truck Speed and Time Loss on Grades.
By Jonathan Jones 1411

Analysis of Unsymmetrical Beams by the Method of Segments.
By Sol Lifsitz 1413

Some Thoughts on Engineering Education.
By William A. Conwell, and A. Amirikian 1415

Effect of Stress Distribution on Yield Points.
By J. F. Baker and J. W. Roderick 1421

CONTENTS FOR DECEMBER, 1946 (*Continued*)

PAGE

Strength of Beams as Determined by Lateral Buckling.
By George Winter, and David B. Hall 1425

Drawdown Test to Determine Effective Radius of Artesian Well.
By Carl Rohwer 1433

Space Resection Problems in Photogrammetry.
By Earl Church 1435

Technical Survey—Brooklyn Bridge After Sixty Years: A Symposium.
By Nomer Gray 1437

Topographic Surveys: Progress Report of the Committee of the Surveying and Mapping Division on Topographic Surveys.
By Paul Van Buskirk 1438

INDEXES

Subject and Author Indexes, 1946 *Proceedings* 1439

A list of "Current Papers and Discussions" may be found on the page preceding the table of contents

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

CLEANING AND GROUTING OF LIMESTONE FOUNDATIONS, TENNESSEE VALLEY AUTHORITY A SYMPOSIUM

PAGE

Foreword 1330

Fort Loudoun Dam.
By ARTHUR L. PAULS, M. ASCE 1332

Douglas Dam.
By T. F. TAYLOR, ASSOC. M. ASCE 1352

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by May 1, 1947.

FOREWORD

Under terms of the basic act and subsequent legislation, the Tennessee Valley Authority (TVA) has designed and constructed sixteen major hydroelectric projects to control and utilize the water of the Tennessee River and its tributaries, thus providing for navigation, flood control, and the generation of electric power. These major projects (see Fig. 1) are located at the most feasible sites to insure the unified development of this river and its tributaries consistent with the economical adaptation of structures to the geology, topography, and hydraulic conditions.

The geology of these sites is interesting. The local formations involved are the Knox dolomite, Chickamauga limestone, and some subordinate shale mem-

FIG. 1.—DAMS OF THE TENNESSEE VALLEY AUTHORITY

bers. In general, both the Knox dolomite and the Chickamauga limestone are fairly soluble when considered over geologic intervals of time, and for this reason numerous seams and cavities abound in the region.

In the construction of these major hydroelectric structures for the development of the valley, an important by-product, especially for the civil engineering profession, has been the mass of technical data made available as a result of the many problems encountered and overcome. A record of the foundation treatment required in the construction of these projects provides one type of technical data which, by its nature, is of particular value to civil engineers. Prior to this Symposium, two groups of papers published in the Society's *Transactions* have described the foundation treatment at Norris,

Guntersville, Chickamauga,[1] Kentucky, and Hales Bar[2] dams. The material included in these papers concerned either the geology or the treatment required at the dam site. The two papers presented herewith serve to complete this special series.

Foundation treatment at the Douglas and Fort Loudoun dams has followed the general procedure developed since 1936 for TVA dams founded on limestone. The success of the treatment illustrates that gravity type dams may be founded on limestone satisfactorily provided that proper methods of excavation, cavity filling, and grouting are employed.

At the beginning of construction the late Theodore B. Parker, M. ASCE, was chief engineer. He was succeeded on July 1, 1943, by C. E. Blee, M. ASCE. Arthur L. Pauls, M. ASCE, was chief construction engineer. E. C. Eckel, Affiliate, ASCE, was chief geologist until his death in November, 1941, when he was succeeded by Berlen C. Moneymaker, Assoc. M. ASCE.

At Fort Loudoun Dam, J. K. Black, M. ASCE, was project manager; and J. F. Partridge, M. ASCE, construction engineer. E. C. McClenagan was construction superintendent.

On Douglas Dam, Lee G. Warren, M. ASCE, was project construction manager; T. F. Taylor, Assoc. M. ASCE, construction engineer and later acting project manager; and James S. Lewis, Jr., M. ASCE, construction superintendent.

Appreciation is expressed to A. H. Weber, civil engineer, for assistance given in the preparation of the first paper in this Symposium.

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[1] "Foundation Experiences, Tennessee Valley Authority: A Symposium," *Transactions*, ASCE, Vol. 106, 1941, p. 685.

[2] "Unusual Cutoff Problems—Dams of the Tennessee Valley Authority: A Symposium," *ibid.*, Vol. 110, 1945, p. 947

FORT LOUDOUN DAM

BY ARTHUR L. PAULS,[3] M. ASCE

SYNOPSIS

Undoubtedly, the most distinctive feature of the foundation at Fort Loudoun Dam was the presence of solution cavities at unprecedented depths below the water table. Solution occurred at depths as great as 300 ft below the original ground surface and 260 ft below the water table. Experience with this foundation has demonstrated that structural deformational features rather than rock solubility often control the depth and extent of weathering. Established grouting procedures and practices are described only briefly to permit a more complete discussion of the deep mining operations in the large bedding cavities both for consolidation and for the construction of an effective cutoff. In the north core wall, cavities were mined to a depth of 220 ft under the most difficult working conditions.

INTRODUCTION

Fort Loudoun Dam is the ninth and final dam, on the Tennessee River, necessary for extending the navigable channel to Knoxville, Tenn.—a distance 650 miles upstream from the confluence of the Tennessee and Ohio rivers at

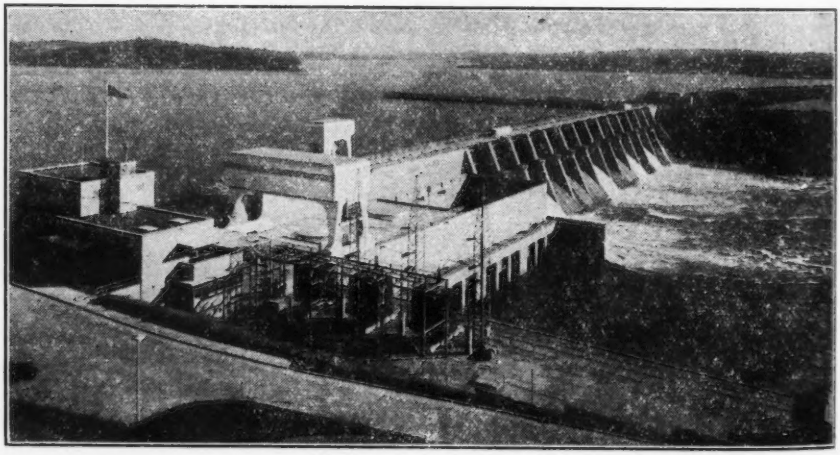


FIG. 2.—FORT LOUDOUN DAM VIEWED FROM THE NORTH BANK OF THE RIVER

Paducah, Ky. (see Fig. 1). The structure is located about 1 mile from Lenoir City, in Loudon County, Tenn.; 602.3 miles above the mouth of the river; 1.2 miles above the confluence of the Little Tennessee River; and 47.7 miles downstream (or 26 miles by highway) from Knoxville.

[3] Alhambra, Calif.; formerly Chf. Constr. Engr., TVA, Knoxville, Tenn.

The over-all length of the combined earth and concrete dam (see Fig. 2) is 4,185 ft; and the structure is 95 ft higher than the original river bed or 165 ft from the lowest excavation. The south embankment is a rolled earth fill dam 2,600 ft long between the lock and the south abutment. The navigation lock between this earth embankment and the spillway has a chamber 360 ft long and 60 ft wide with a maximum single lift of 80 ft. The spillway section has fourteen bays, each 40 ft wide closed with Tainter gates 32 ft high. The powerhouse and intake span the original main river channel and have an over-all length of 280 ft in four bays. The service bay is stepped into the north bank and a core wall extends about 300 ft into the north abutment. A saddle dam on the north rim, 2,283 ft upstream from the main dam, is a rolled fill structure 525 ft long.

Selection of Site

As early as 1927, geologic studies were made by the Army Engineers in the vicinity of Fort Loudoun Dam to determine the availability of a suitable dam site. Investigations were continued by the TVA, and many sites were explored within the 10-mile to 12-mile reach of the river defined by the operating level of Watts Bar Dam downstream and by the cost of dredging the upper reaches of the Watts Bar pool as balanced against the cost of a higher dam. The occurrence of major faults and other geologic disturbances in the area limited the number of usable sites both topographically and geologically. Furthermore, the underlying rock of the entire area is more or less calcareous and therefore subject to weathering and solution.

Seven sites were studied, and four of these were explored by diamond core drilling. The Belle Canton Island site at mile 602.3 was finally selected; and, although this site was obviously not the best topographically, it did appear to be geologically preferable to any of the other sites available (with two exceptions, both of which had other disadvantages that offset the more suitable

TABLE 1.—Drilling and Grouting Quantities

Feature	Drilling (Lin Ft)			Grout (cu ft)	Concrete (cu yd)
	Diamond	Wagon	Calyx		
South embankment	86,557	57,588	4,601	68,500	1,395
Lock	103,405	99,674	1,282	97,128	405
Spillway	122,837[a]	38,364	3,703	127,510	3,277
Powerhouse and intake	63,929[a]	1,600	1,377	24,839	1,196
North abutment	31,093	...	4,044	117,781[b]	3,543
North reservoir rim	34,713	...	...	408,976[b]	...
Saddle dam	3,305	...	...	222,653[b]	...

[a] Includes drain holes. [b] Includes clay content in cement grout.

foundation). Enough drilling was completed at the Belle Canton Island site to disclose that the weaknesses could be corrected although the magnitude of the problem, as shown by the drilling, grouting, and concreting quantities in Table 1, was fully recognized when treatment was begun.

Geology

The topography of the Fort Loudoun dam site was not as favorable as that at the other sites because the alluvial valley is unusually wide—more than 2,800 ft (see Fig. 3). The river channel, approximately 1,800 ft from bank to bank, was originally divided by three alluvial islands into four separate channels from 100 ft to 300 ft wide. At the north abutment of the dam, the main channel was cut into the base of the ridge that formed the right bank of the river along the apex of a broad meander beginning just upstream. In the process of cutting into the right bank, this meander receded from the higher ground leaving a clay-covered terrace slope which forms the south abutment of the dam. The topography ranges from El. 730 in the river bed to El. 900 on the crest of the flanking ridges.

The only bedrock exposed at the dam site was that in the bluff on the north abutment ridge. Elsewhere the rock was covered by overburden varying in

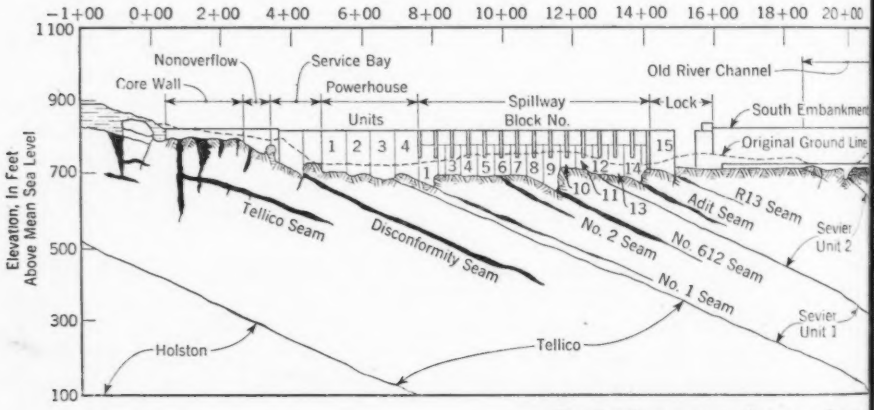


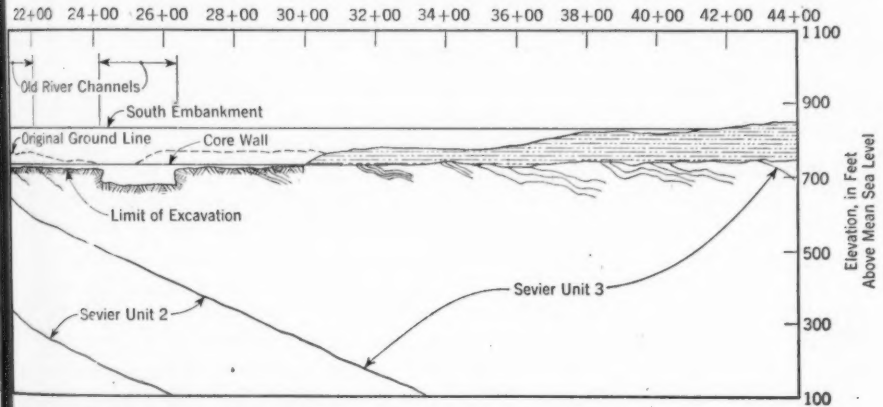
Fig. 3.—Generalized Geologic Section

thickness as much as 40 ft and consisting of both alluvial and residual deposits, although the alluvial material capped most of the area where outcrops were not present. Stratigraphically, the underlying rock belongs to the Ordovician period of east Tennessee. Of the belt of rocks in the vicinity, only the Tellico and Sevier formations are actually involved in the dam foundation.

The Tellico formation is generally regarded as sandstone, but in this belt it is an arenaceous limestone; and, in spite of the siliceous character of the formation, it is nevertheless soluble. Because of the irregularity of the bedding and the presence of less soluble wavy stylolitic bands, cavities are not readily formed along the bedding planes. A distinctive feature of this formation at Fort Loudoun is the presence of an intraformational disconformity approximately 100 ft below the contact of the Sevier formation. There was an old erosion surface along the bedding at this position on which from 3 ft to 5 ft of shaly and fossiliferous limestone were deposited. This old erosion surface had been further attacked by solution, and a cavity was present along it.

The Sevier formation as a whole is quite diversified—ranging from a coarsely crystalline marble, through limestone and shaly limestone, to a dense blue-gray calcareous shale. It has been found practicable to divide this formation into three units (see Fig. 3), each with a predominant lithologic type. Unit 1, or the lower 225 ft, is predominantly limestone. Near the top of this unit the amount of shale becomes progressively more abundant and grades into the part designated as unit 2. As a whole, this unit is more soluble than the remaining part of the Sevier formation; hence, the cavities are usually larger and are either filled or partly filled with a clay residue. Unit 3 is mostly a gray calcareous shale; but, as in the other two units, there is a certain amount of interbedded shaly limestone varying in thickness from 20 ft to 100 ft. As this unit is less resistant structurally than the lower units, it has suffered considerable deformational effects and consequently is deeply weathered.

The geologic structure at the site is typical of the regional structure with a strike of north 40° to 45° east and an average dip of from 30° to 35° southeast.



SHOWING THE LOCATION OF MAJOR SEAMS

Since the base line of the dam makes an angle of almost 67° with the strike, the rock dips toward the south embankment but slightly upstream in relation to the structure. Although there are no major faults in the dam foundation, numerous minor faults and bedding plane slips exist. There is no evidence of any extensive movement along these slip planes; the movement is merely a readjustment to the forces attending the major thrust faulting and folding in this region. A thrust fault in the Tellico formation is unique in that it is the only thrust fault of any magnitude at the site that was not developed along the bedding. However, the movement is not considered extensive although a cavity has been formed along this weakness. Almost without exception, the many solution cavities developed in the foundation were controlled by solution along geologic structures—faults, joints, and bedding planes. At the intersection of these fractures, the larger and more continuous cavities and seams were developed. Solution followed intricate patterns with channels and cross channels predominating, separated at irregular intervals by rock pillars. The

presence of these pillars greatly augmented the bearing strength of the formation, but a certain amount of consolidation grouting was considered necessary. Contrary to the normal conception that relatively more cavities are to be expected in the purer limestone, the cavities in this area were primarily controlled by structural features rather than by the purity of the rock. Also, founding a dam on impure, less soluble, limestone does not necessarily insure a better foundation than one on a pure, more soluble limestone—provided the latter is free of deformational features.

For convenience in the field but of no regional geologic significance, the master cavities or weathered bedding seams were designated from lower to upper as the Tellico, disconformity, No. 1, No. 2, No. 612, adit, and R13. These seams and their positions relative to the dam are shown in Fig. 3.

Formation and Character of Cavities

An excellent opportunity was presented during foundation excavation and treatment at Fort Loudoun Dam for the study of solution cavities both above and within the zone of saturation. Subwater table cavities were explored and treated at what were considered unprecedented depths. Only in recent years has much significance been placed on the formation and occurrence of cavities in the deep zones of saturation. Many formerly contended that all cavities existing at substantial depths below the prevailing water table must have originally been formed at or near a former water table level and that either the cavities had been lowered by a regional rock subsidence or a former water table had been raised to engulf the previously formed cavities. Many cavities and caves now situated within the zone of saturation had such an origin. However, in the Appalachian Valley where it is evident that the crustal movement has been one of uplift rather than subsidence, and where there are no old buried river channels below present stream levels, it is apparent that the cavities developed below the water table are the results of subwater table solution and that this development is a normal geologic process. The fact that there may be deep underground flow in the zone of saturation is entirely compatible with hydraulic principles—the requisite being that the point of exit be at a lower elevation than the point of entrance, but not necessarily as low as the deepest zone of flow.

At Fort Loudoun Dam the most obvious explanation of the deep subwater table flow is that the ground water (1) finds its way into the exposed ends of the previously formed channels developed along such structural weaknesses as bedding planes, faults, and joints; (2) starts its migration along the strike and down the dip of these conducting channels in irregular circuitous paths to a depth determined by the hydraulic head, by the solubility of the rock, and by the structural weaknesses; and (3) resurges upward to the point of discharge. The flow and resulting creation of cavities proceed downward until an upward or lateral movement is less resistant than continued downward flow. Thus, a system of both vertical siphons and low-angle inverted siphons develops, one group below the other, the depth dependent primarily on the differential head. Often, the watercourses are clogged with insoluble residue that causes the water to seek deeper levels as solution continues. The existence of an inverted siphon

flow was actually observed during excavation and was further attested by the presence of cold-water springs or boils in the river bed. Dye injected into holes upstream from the cofferdam showed in the boils encountered in the upper few feet of excavation more than 1,000 ft away.

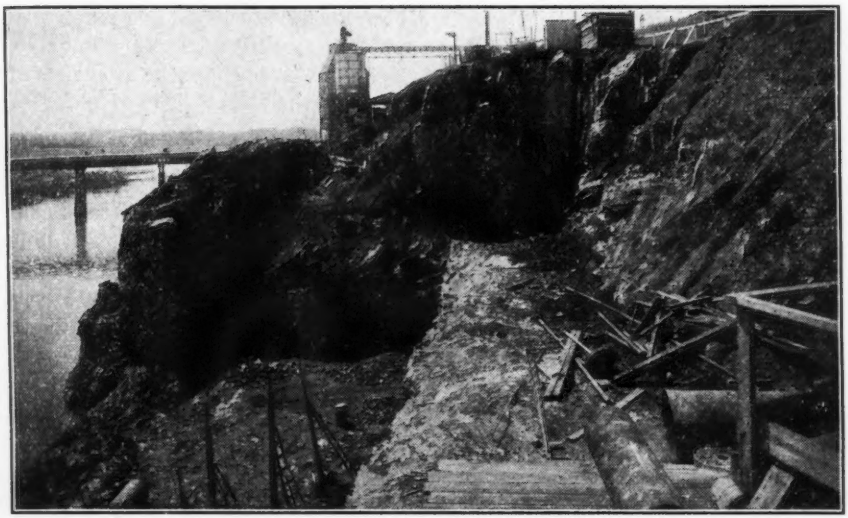


FIG. 4.—NORTH ABUTMENT, SHOWING TYPICAL ROCK PINNACLES

In the north abutment, solution has resulted in the formation of a series of deep troughs separated by intervening pinnacles (see Fig. 4). This formation, occupying a ridge with a steep bluff at the encroaching meander of the river, demonstrated an interesting point—namely, that the presence of a steep and apparently sound limestone rock bluff is not a guarantee that sound rock extends backward into the bluff; it may be nothing more than a false front with deep weathering just behind it.

FOUNDATION INVESTIGATION

For purposes of exploration the pattern of core drilling consisted of holes on lines parallel to the old preliminary drill ranges to enable the utilization of holes previously drilled. Holes were spaced on 50-ft centers with some intermediate check holes. Ordinarily, 3-in. diamond drill bits were used. The depth of holes depended on the geologic structure and ranged from 33 ft to 198 ft below the original rock surface.

For the lock location, ten ranges were drilled from upstream to downstream—a total of ninety-one holes and 8,580 lin ft. In addition, one 36-in. calyx drill hole 75 ft deep was sunk for visual inspection of the rock in place. Exploratory holes were drilled concurrently with construction, and each section was drilled in advance of excavation to determine the problems involved. A total of 58,193 lin ft of diamond drilling and 365 lin ft of calyx drilling was

completed for the exploration of the dam foundation. (A general description of calyx drilling has been presented elsewhere by Mr. Lewis.[4])

South Embankment

The bedrock of the south embankment, predominantly calcareous shale of unit 3 of the Sevier formation, was highly weathered along bedding plane slips from El. 730 to El. 690. The bedding cavities were filled with soft yellow clay, which was the residuum of weathering. Two old river channels, as indicated in Fig. 3, were extremely cavernous throughout the upper 40 ft of rock. A typical weathered section is shown in Fig. 5.

A trench was excavated to fairly sound rock from the lock southward along the line of cutoff to Station 29+63. It was approximately 3 ft wide; and, after 4-in. vertical pipes were set on 2-ft centers, the trench was backfilled with concrete to a depth of 5 ft. Wagon drill holes were drilled in 20-ft, 30-ft, and 40-ft stages on 2-ft centers using the 4-in. pipe for casing. After each stage of drilling, the holes were thoroughly washed using pressures of 20, 30, and 40 lb per sq in., respectively, for each of the three stages, followed by grouting under similar pressures. Some surface leakage of grout occurred during each stage but it appeared farther upstream and downstream for each subsequent stage—a good indication that a cutoff was being obtained as the holes were deepened. The effectiveness of the cutoff was tested by grouting 60-ft diamond drill holes

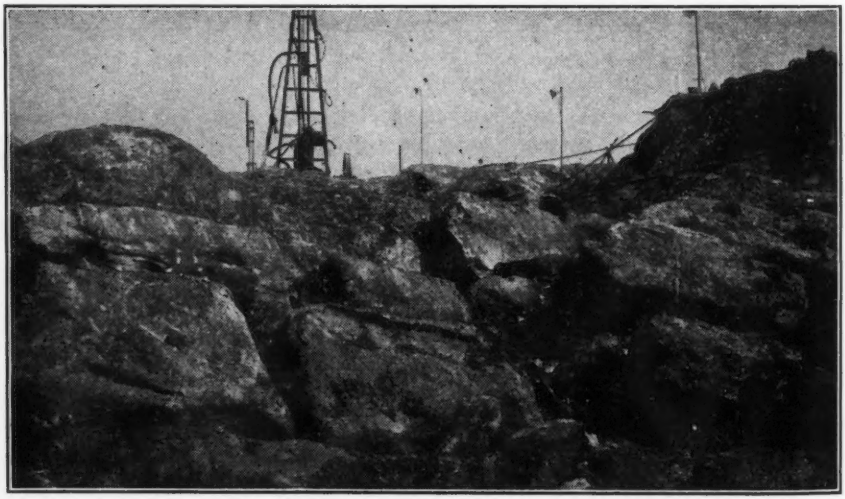


Fig. 5.—Weathered Rock in the Old River Channels; South Embankment

on 10-ft centers. Very little additional grout was accepted by these holes, and the cores recovered indicated excellent grout penetration along the weathered shale beds.

[4] "Large Core Drills Aid Construction at Chickamauga Dam," by James S. Lewis, Jr., *Transactions* ASCE, Vol. 105, 1940, p. 850.

Rather than attempt to treat the intersection of the cutoff with the old river channels in a similar manner, a positive cutoff was secured by backfilling the 38-in. calyx drill holes with concrete. Originally, these holes were drilled to El. 690 on 5-ft centers with the intervening unsound rock being removed. Because the practice proved both expensive and dangerous, the holes were backfilled with concrete and an intermediate overlapping 38-in. hole was drilled and concreted. Any unsound concrete that might have been placed was checked by grouting through wagon drill holes in the center of each 38-in. hole. Seams below El. 690 were grouted through diamond drill holes on 10-ft centers and drilled to El. 660. Where mining between the 38-in. holes on 5-ft centers was possible, wagon drill holes on 2-ft centers were grouted to form the intervening cutoff and to consolidate any unsound concrete. While this cutoff was being constructed, sufficient water entered the holes to prohibit drilling until the entire area was encircled with diamond drill holes on about 5-ft centers 20 ft upstream and downstream, and grouted using neat cement with calcium chloride added to accelerate the set.

Permeability and porosity tests indicated that the overburden south of the core wall was essentially impervious and that a satisfactory cutoff could be obtained by stage grouting. Holes were first drilled 20° from the vertical to intersect any strike joints present; but, since there was a relatively small grout acceptance, the remaining holes, for economy, were drilled vertically on 6.25-ft centers and grouted in 10-ft stages to El. 680 with deeper check holes to El. 640 every 50 ft. The first stage was grouted at a pressure of 0.5 lb per foot of overburden and each successive stage at an additional 5 lb per foot of overburden until the last stage, which was grouted at a pressure of 50 lb per sq in.

Lock

In most of the rock underlying the lock river wall and the downstream half of the land wall, closely spaced weathered seams and cavities were present in depths from 30 ft to 40 ft. Below this zone the cavities were separated by from 20 ft to 50 ft of intervening sound rock that possessed considerable bearing strength in the form of irregularly spaced rock pillars. As the major cavities in the lock foundation—those along the bedding plane slips—were inclined to the southeast with an upstream pitch, they became progressively nearer the rock surface toward the downstream end of the lock.

Obviously, with so many deep cavities and seams existing under the lock area, it was not practicable to excavate the entire zone of cavitation. If a stratum of sound rock were deep enough, although superimposed on defective material, it could support the lock structures through arch action; and, as a precautionary measure, the entire lock area was subjected to consolidation grouting prior to cutoff treatment—in most instances, before concreting operations. Even with this program of foundation treatment, excavation to depths as great as 60 ft was necessary for the river wall foundation.

The depth and spacing of the consolidation holes were contingent on rock conditions and foundation requirements. Both first-stage holes 20 ft deep and second-stage holes 40 ft deep were sunk with wagon drills on 10-ft and 5-ft centers in parallel lines from 10 ft to 12 ft apart. Stage one was usually grouted

at a pressure of 20 lb per sq in.; and stage two, at a pressure of 40 lb per sq in. The program was supplemented with diamond drill check holes, and grout pipes were calked into any open seams for grouting after concrete was poured, as shown in Fig. 6. Under the river wall and lower miter sill, the grouting program was supplemented by mining through calyx drill holes wherever necessary to secure a cutoff and to augment bearing strength.

The cutoff across the lock consisted essentially of a shallow curtain of wagon drill holes 40 ft deep grouted at low pressure and a deeper curtain of diamond drill holes from 100 ft to 150 ft deep grouted at high pressure. The low-pressure curtain was designed primarily to reduce, to a minimum, the percolation under the structure through small cracks and joints in the rock surface. Stage one of the low-pressure holes was drilled 20 ft deep on 2-ft centers and grouted at a pressure of 20 lb per sq in. The holes were redrilled to 30 ft, washed, and then grouted at a pressure of 30 lb per sq in. They were finally drilled to 40 ft, washed and grouted at a pressure of 40 lb per sq in. Diamond drill holes for the high-pressure curtain followed the same line as those for the low-pressure curtain and varied in spacing and depth to satisfy structural and geological conditions. In highly weathered areas, grouting alone was not considered adequate and 36-in. calyx drill holes were spaced on such centers that it was possible to mine out unsound material between the holes and to backfill the holes with concrete. Because of the soundness of the near-surface rock under the upper miter still, the low-pressure grout curtain was eliminated and the high-pressure curtain was utilized to grout the deep bedding seams.

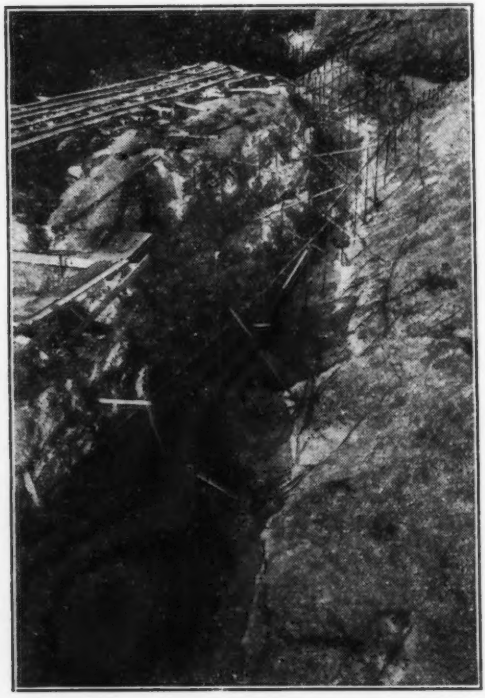


FIG. 6.—PIPES SET IN LOCK FOUNDATION FOR CONSOLIDATION GROUTING AFTER PLACING CONCRETE

SPILLWAY AND POWERHOUSE

Limestone of Sevier unit 2 underlies the spillway from its intersection with the lock to the 9–10 block line where it grades into unit 1 for the remainder of the spillway area to the 1–2 block line. At this line a gradational contact

with the Tellico formation underlying the powerhouse takes place, see Fig. 3. Exploratory drilling had shown the rock of the spillway to be intricately dissected by weathered seams and cavities from the top of rock at El. 730 to El. 700. Below this zone of weathering, the rock was essentially sound except for the presence of several master bedding cavities that were remarkably continuous and extended to great depths.

The No. 1 seam, the No. 2 seam, the No. 612 seam, and the adit seam (Fig. 3) were the structural features that controlled the spillway excavation pattern. Other structural weaknesses were comparatively tight except at or near the intersection of bedding cavities and faults. As in the lock, some compromise had to be made between open excavation and a program of foundation treatment. Excavation was generally carried to El. 700 (El. 712 for the apron), and the additional rock was excavated down the dip of these major bedding seams to a point where mining and grouting were more economical than was the process of making a further open cut. The resulting spillway excavation had the appearance of a level rock surface modified by the removal of large wedge-shaped blocks at the outcrop of the major bedding planes.

Except for the disconformity seam which underlies practically the entire powerhouse area and several minor strike joint cavities, the foundation rock of the powerhouse was only slightly weathered and little excavation beyond design requirements was necessary. The intake section was excavated to El. 715 with additional rock removal along the weathered bedding planes in intakes 2 and 3 and with removal of weathered rock at the outcrop of the No. 1 seam in intake 4. Excavation for the draft tubes was governed by design with the exception of a wedge-shaped block along the outcrop of the disconformity seam in Sevier unit 1.

Consolidation grout holes were drilled throughout the entire spillway area to augment the bearing strength of any unsound rock. Consolidation consisted not only of grouting through wagon drill holes and diamond drill holes, but also of mining large seams and cavities through calyx drill holes and backfilling these mined zones with concrete. The most extensive mining operation for consolidation was along the adit seam under spillway block 15. The full width of the block, upstream and downstream, was excavated by undermining; and, because of the overburden, the operation was accomplished in stages. A section 10 ft wide was first excavated and a concrete pillar was placed for support. This procedure (Fig. 7) was then repeated until the entire area had been mined out and backfilled with concrete.

Consolidation grouting of the intake sections was a simple procedure compared with the extensive and intricate program necessary under most of the spillway blocks. Excavation left only the deep disconformity seam to be treated, and this was accomplished through a uniform pattern of diamond drill holes on 10-ft centers grouted at a pressure of 50 lb per sq in. Occasionally, holes were taken to the deeper Tellico seam but it was found to be essentially tight.

All areas of unsound rock which might be potential aquifers for leakage under the spillway and powerhouse were treated by constructing a cutoff curtain extending the full length of the structure along the upstream face. The

method of treatment varied to meet the conditions as they were encountered. It consisted of stage grouting through diamond drill holes and wagon drill holes, and of mining and backfilling with concrete through calyx drill holes—in those areas in which grouting alone was not sufficient. Mining involved the

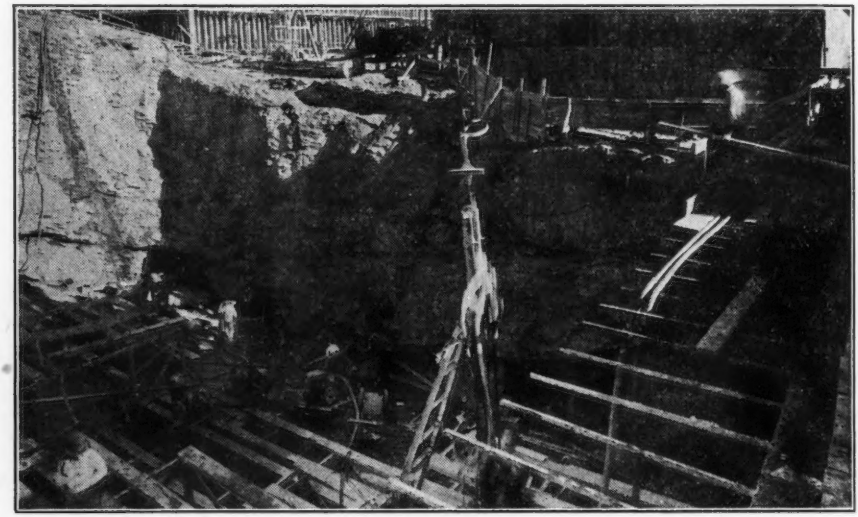


FIG. 7.—MINING ADIT SEAM UNDER SPILLWAY BLOCK 15

removal of a part of the unweathered rock immediately surrounding the actual seam to provide a tunnel large enough for a man to work. Most of these cutoff tunnels were about 4 ft high and from 4 ft to 5 ft wide. On completion of the mining program, the tunnel was backfilled with concrete and the calyx drill holes were filled with concrete to the surface to secure a hydrostatic head on the tunnel concrete that greatly reduced the possiibility of voids and air pockets.

Exploratory drilling had shown the No. 612 seam (Fig. 3) to be badly weathered throughout most of the spillway section. Eight calyx drill shafts were started in spillway blocks 10, 11, and 12, but their completion was prevented by a prohibitive amount of water that could not be controlled successfully by pumping. An auxiliary cutoff was grouted across block 10 along the strike to prevent this leakage and to form a barrier against which a cutoff could be grouted. Holes were drilled and grouted first on 16-ft centers, followed by holes on 8-ft centers and 4-ft centers, and finally on 2-ft centers throughout blocks 10 to 14. The entire procedure was repeated in three stages—drilling and grouting the same holes first on 8-ft centers, then on 4-ft centers, and finally on 2-ft centers. On completion of this work, the calyx drill holes and mining operations were completed without further water problems.

Low-pressure grouting, through wagon drill holes, was applied to spillway blocks 12, 13, and 14 only. The remainder of the spillway was subjected to closely spaced deeper diamond drill holes grouted in stages at a pressure of

100 lb per sq in. Final spacing of holes was usually 2 ft on centers; and, in the powerhouse area, holes were spaced on 4-ft centers. Fig. 8 shows the pat-

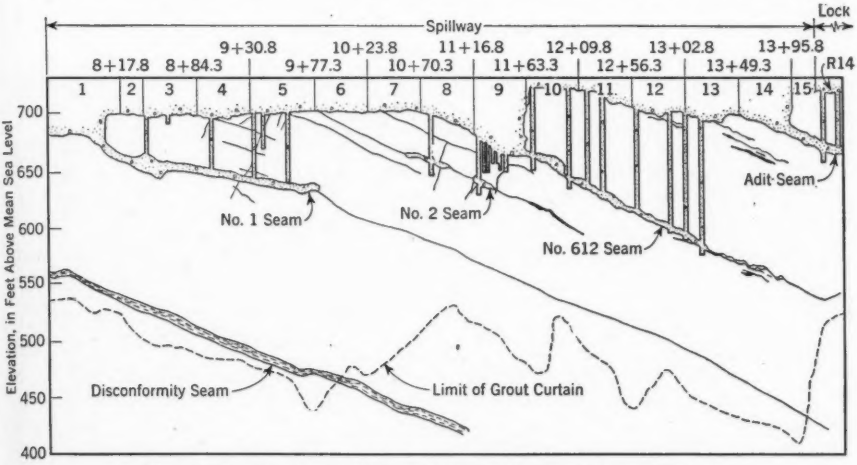


FIG. 8.—SECTION ALONG SPILLWAY CUTOFF SHOWING TREATMENT OF THE LARGE BEDDING CAVITIES

tern of cutoff treatment across the spillway area and Fig. 9 shows similar treatment given to the powerhouse foundation.

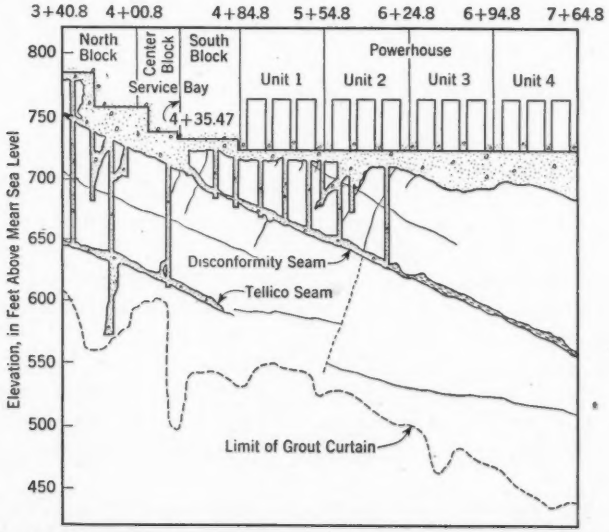


FIG. 9.—SECTION ALONG POWERHOUSE AND SERVICE BAY CUTOFF SHOWING TREATMENT OF LARGE BEDDING CAVITIES AND STRIKE JOINTS

Although it was not part of foundation treatment, an extensive grouting program undertaken for sealing leakage under the cofferdam was largely responsible for controlling the flow of water into the river bed construction area.

The first-stage cofferdam enclosing the lock and the ten adjoining spillway blocks was essentially tight; but, as excavation progressed below the overburden, several boils developed along the outcrop of the weathered bedding seams and from the diamond drill holes that penetrated deeper cavities. As further leakage was anticipated with deeper excavation, the entire perimeter of the cofferdam was subjected to grouting—first through holes on 20-ft centers and then through holes on 10-ft centers with some holes on 5-ft and 2.5-ft centers. Initial grouting was done with neat cement; but, since large quantities were consumed in the first holes, clay-cement grout was substituted for the major part of the work. For the more persistent leakage, asphalt was used successfully to stop the flow of water.

A similar grouting program was developed for the second-stage cofferdam comprising the remainder of the spillway and powerhouse area, but grouting of this stage was completed prior to unwatering. Although the second-stage cofferdam was tighter than the first-stage cofferdam and presented practically no water problems, its imperviousness was not completely ascribed to the grouting of the cofferdam before unwatering. The large quantities of clay-cement and asphalt injected into the first-stage cofferdam had considerable effect on the tightness of the second-stage cofferdam as was revealed by the excavation. Drilling and grouting quantities for the cofferdams are listed in Table 2.

TABLE 2.—DRILLING AND GROUTING QUANTITIES FOR THE COFFERDAMS, FORT LOUDOUN DAM

(Spillway and Powerhouse Areas)

Cofferdam	Drilling (lin ft)	Grout (lin ft)	Asphalt (cu ft)
Stage one	42,850	668,940	9,408
Stage two	26,240	157,090	500

NORTH ABUTMENT

Exploration of the north abutment revealed that the depth of excavation required to reach sound rock would necessitate a nonoverflow gravity type of dam between the service bay and the north core wall. In a manner typical of the entire north abutment area, the Tellico formation underlying the proposed nonoverflow section and service bay was dissected by a complex series of vertical joints along which weathering had taken place to great depth, leaving high pinnacles of comparatively sound rock separated by deep troughs filled with a sandy clay residuum (see Fig. 4). This condition was further aggravated by the presence of the disconformity seam (Fig. 3) which outcropped in block 3 of the nonoverflow section and necessitated a deeper excavation than was anticipated.

Under the service bay, the disconformity seam was badly weathered and the foundation was excavated to this seam over most of the service bay area. With this depth of excavation, the only consolidation necessary under the service bay consisted of grouting through diamond drill holes on 10-ft centers along ranges 20 ft apart throughout the three upstream blocks. Consolidation grout-

ing was supplemented by driving a 4-ft tunnel along the unexcavated part of the disconformity seam and filling it with concrete, as shown in Fig. 10.

Since it was impracticable to remove the rock to the bottom of the several deep vertical joints encountered in the nonoverflow section, excavation was

FIG. 10.—DISCONFORMITY SEAM AND ASSOCIATED STRIKE JOINT IN THE SERVICE BAY

stopped at a depth considered adequate for bearing strength. A cutoff was constructed by mining all unsound areas through deep calyx drill holes, backfilling with concrete and subsequently grouting through closely spaced diamond drill holes. As soon as excavation in nonoverflow blocks 1 and 2 was completed, concrete was poured to El. 800 to improve working conditions and to prevent the sliding of material into the cut. Barrel forms, as shown in Fig. 11, were brought up through the pour at several points to reduce later calyx drilling.

The seams treated were of two major types—(*a*) the shallow vertical weathered joints not removed by excavation, and (*b*) the deeper continuous Tellico seam with associated strike joints. Shallow treatment in block 1 of the nonoverflow section was unnecessary as all rock below the disconformity seam was essentially sound. In block 2, several weathered joints were mined through calyx drill holes. These holes were later deepened for mining the Tellico seam. Excavation to the disconformity seam in block 3 revealed a series of shallow but closely spaced badly weathered joints; and, instead of mining between holes, 36-in. calyx drill holes were first sunk on 5-ft centers and backfilled with concrete; intermediate holes were drilled to overlap the first holes. Similar treatment was given throughout block 4 with three holes later deepened to the Tellico seam.

Treatment of the Tellico seam under the nonoverflow and service bay section saw a difficult operation. Not only did the seam occur at depths as great

as 150 ft, but its treatment was complicated by the presence of weathered strike joints that required additional and deeper mining. Fig. 12 will assist the reader in the discussion to follow. Three calyx drill holes, about 5 ft apart, were

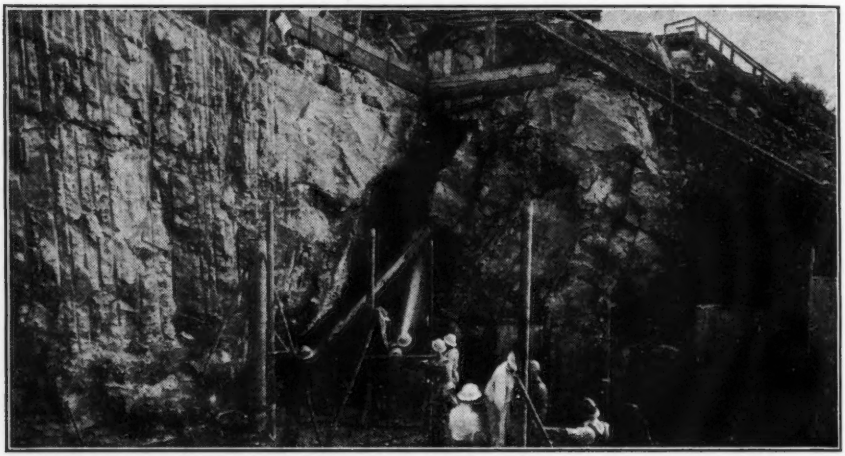


FIG. 11.—PLACING BARREL FORMS AND 4-IN. CASING FOR LATER DRILLING

drilled in the nonoverflow section with the intention of mining the cavities between holes. Between El. 743 and El. 723, in a hole at Station 2+13, a strike joint was encountered; it was thoroughly mined and a concrete plug was placed to El. 745. The plug was then drilled out and drilling was con-

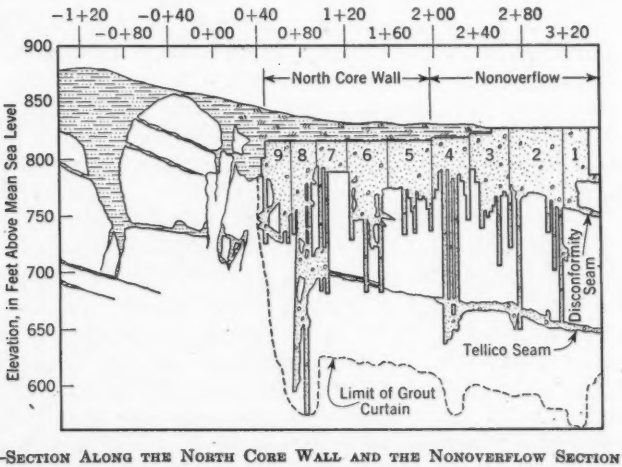


FIG. 12.—SECTION ALONG THE NORTH CORE WALL AND THE NONOVERFLOW SECTION CUTOFF

tinued to the Tellico seam where excessive water entered the excavation. Concrete plugs were poured in the calyx drill holes, and a ring of six diamond drill holes, grouted at high pressure, was effective in stopping the flow of water.

Preparatory to tunneling southward along the Tellico seam, the steep wedge-shaped cut, caused by mining the strike joint, was poured to El. 665 to provide a level floor from which to work. The tunnel was driven along the Tellico seam southward until it intersected a previously drilled hole at Station 2+75. A weathered strike joint at Station 2+70 was mined below the Tellico seam to El. 650 where weathering had ceased; but, to eliminate the possibility of any deeper solution along the joint, six shallow diamond drill holes were drilled from the tunnel floor along the down-dip side of the joint and grouted.

Another calyx drill hole intersected the Tellico seam at Station 3+12, and mining was started northward to intersect the first section of the tunnel. Excellent progress was made in this section despite the presence of a weathered bedding seam intersecting the Tellico seam at Station 3+12 and pinching out at Station 2+63. By using the Tellico seam as the floor and the bedding cavity as the roof of the tunnel, a cutoff was obtained along both these structures in a single operation. After the tunnels were joined, they were cleaned and backfilled with concrete.

Concrete was placed in such a manner that only a minimum of redrilling was necessary in any hole to be used for further mining. Treatment of the Tellico seam by mining north of Station 2+08 was not required; however, the unmined area was subjected to grouting. Before tunneling the Tellico seam southward under the service bay from Station 3+12, the area was subjected to a grouting pattern to check the flow of water that was indicated by exploratory drilling. Three calyx drill holes were bored to the seam. The area was found to be more badly weathered than had been anticipated; therefore, mining was continued to Station 4+72, as indicated in Fig. 9. Several strike joints in the service bay area were also intersected and treated similarly to those encountered in the nonoverflow section.

Following the mining operation, the entire nonoverflow and service bay were supplemented with grouting through diamond drill holes on 2-ft centers throughout the service bay and blocks 1 and 2 of the nonoverflow section Blocks 3 and 4 of the nonoverflow section were treated with holes on 4-ft centers.

North Core Wall

Rock conditions under the north core wall were similar to those under the nonoverflow section. However, the deep weathering was largely confined to two major areas separated by a ridge of comparatively sound rock. The first of these areas was under blocks 5 and 6 and the second—by far the most deeply weathered part of the entire dam foundation—existed under blocks 8 and 9 (see Fig. 12). Exploratory drilling disclosed a nearly vertical oblique joint striking normal to the dam axis and weathered to the unprecedented depth of approximately 220 ft. This joint had been subjected to some lateral movement, thought to be local, but possibly with a direct bearing on the depth and degree of the solution.

An open cut excavation was first made to El. 800. Further excavation to bare the rock for deep drilling and foundation treatment was done by trench-

ing, and heavy timbering was required to effect this operation. A trench 4 ft wide was dug to El. 745 at the south end and to El. 758 at the north end. Below these levels, weathering was restricted to the narrow vertical joints which were mined out through calyx drill holes and backfilled with concrete. Before drilling was begun, the bottom of the trench was filled with concrete to El. 800 and El. 796 at the north end and south end, respectively. Barrel forms were again set as required to reduce the drilling footage. Treatment under blocks 5, 6, and 7 of the core wall consisted of both shallow and deep calyx drilling. In some areas a shallow cutoff was obtained with overlapping 36-in. calyx drill holes spaced on 2.5-ft centers. Although this method was possible for shallow cutoffs, it was not feasible for treating deeper cavities because of the large amount of drilling and the difficulty, at greater depths, of obtaining the alinement necessary.

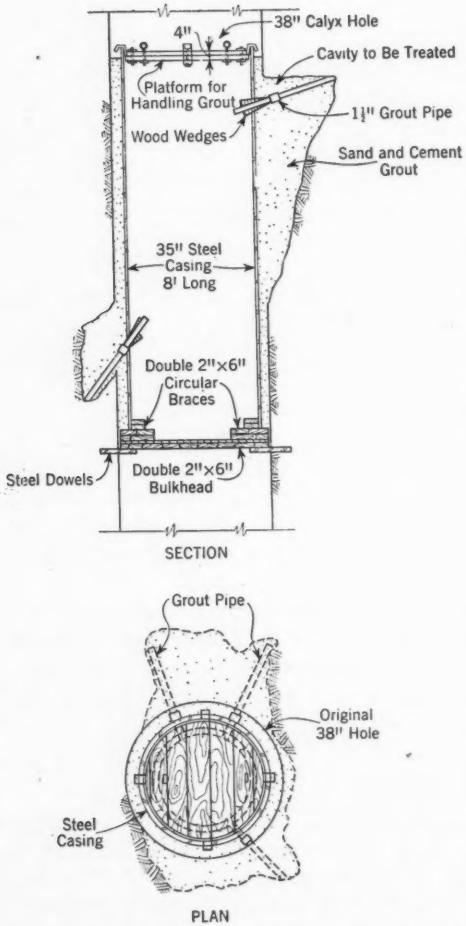


FIG. 13.—METHOD OF CALKING A CAVITY THAT INTERSECTED THE CALYX DRILL HOLE

The most difficult foundation problem—not only of the north core wall but also of the entire project—was the deep vertical oblique joint under block 8. As in other parts of the foundation, calyx drill holes provided shafts for the mining operation. The first of these holes was sunk at Station 0+86.8 (see Fig. 12); and, at El. 712, the hole intersected a highly weathered area at the intersection of the oblique joint and the Tellico seam. Another hole was drilled 10 ft southward at Station 0+96.8, and the cavity was cleaned between these two holes to El. 703 and filled with concrete. The concrete plugs thus formed were redrilled, and mucking was continued to El. 695 where weathering once again was confined to the vertical joint; and the cavity was again filled with concrete. The hole at Station 0+96.8 was then no longer required and was backfilled with concrete.

The mucking, backfilling, and redrilling processes continued in 7-ft stages in the remaining hole to El. 674 where the water inflow became excessive. Another hole at Station 0+76.8 was then drilled to a slightly lower elevation and connected to the cavity with jackhammer holes. A pump was installed to handle the water, and mucking was continued. One hole in the rock was kept open for a sump, by offsetting if necessary, and the other hole was used for mucking.

At El. 635 a slide forced a temporary abandonment of the treatment until the seam-filling material was consolidated by grouting. Preparatory to grouting the seam through diamond drill holes, several minor joints intersecting the walls of the hole and the oblique joint had to be plugged to avoid loss of grout. These joints were calked by inserting a 35-in. steel casing into the calyx drill hole at the point to be treated and hand packing grout into the seam between the casing and the periphery of the hole, as shown in Fig. 13. In addition, nipples for pressure grouting were calked into the seams wherever possible. Four 1.5-in. grout pipes were driven into the seam where the slide occurred and a 4-ft concrete cap was placed. After grouting through these pipes at a pressure of 100 lb per sq in., jackhammer holes were drilled from the calyx drill hole into the seam at several elevations and grouted at high pressures. Concurrently, two ranges of diamond drill holes, one from 10 ft to 40 ft upstream at Station 0+76.8 and the other a like distance downstream at Station 0+86.8 were drilled normal to the core wall for grouting the seam. Holes were drilled on 10-ft, on 5-ft, and, in some places, on 2.5-ft centers and grouted at high pressure.

When the concrete plug was drilled out of the calyx drill hole, the sandy clay residuum had been consolidated, and the process of treating the cavity was continued to El. 593 where it pinched to a fraction of an inch. Deeper treatment was accomplished by placing 1,065 cu ft of grout into two perforated pipes driven in the seam and by injecting 105 cu ft of grout through jackhammer holes drilled into the seam from the adjacent calyx drill hole. A 4-in. pipe was encased in the concrete backfill, and a later diamond drill hole, bored into the seam to El. 478.5, was grouted with 787 cu ft of cement at a surface pressure of from 100 lb to 120 lb per sq in. as the final treatment. Supplementing the calyx drill hole procedure, diamond drill grout holes were drilled on 4-ft centers for the entire length of the cutoff to consolidate any possible concrete shrinkage in the mined areas and to treat minor cracks and joints.

North Reservoir Rim and Saddle Dam

The bearing of the ridge forming the north reservoir rim parallels the strike of the geologic structure with the dip of the rock toward the reservoir. Normally, such a structure would provide a satisfactory foundation; but the presence of a complex pattern of weathered strike and dip joints made a certain amount of rim treatment necessary to prevent leakage downstream or seepage northward through the ridge separating the reservoir from Muddy Creek Valley.

The north rim was treated by placing a grout curtain extending from the end of the core wall upstream more or less along the strike of the rock a dis-

tance of about 800 ft. Diamond drill holes, inclined 45° upstream and northeast to secure a maximum intersection with weathered dip joints, were grouted on 20-ft centers. These holes were reinforced by grouting vertical holes on 10-ft centers in the vicinity of the north core wall and on 20-ft centers farther upstream. Except for the vertical holes near the core wall (which were grouted with cement) the north reservoir rim was grouted exclusively with a clay-cement mix consisting of three parts clay to one part cement.

Drilling was discontinued whenever a cavity or group of cavities was encountered, and the hole was grouted before further drilling. The processes of drilling, grouting, and redrilling were continued throughout the length of the cutoff curtain until the final drilling of each hole showed no water loss and revealed a considerable margin of sound rock below the last cavity. For the upper 440 ft of the curtain, only two to four stages of grouting were necessary, but deeper weathering (to El. 590) toward the north core wall required grouting in as many as eight stages. Although the multiple-stage grouting procedure required a larger quantity of grout than would probably have been used with the single-stage method, the method afforded positive proof of the effectiveness of the grout curtain.

Only moderate treatment of the saddle dam was necessary because of the deep layer of overburden, the dip of the rock toward the reservoir, and the low head to which the structure was subjected. Grout holes were drilled along a zone of comparatively sound rock parallel to the axis of the saddle dam and at an angle of 45° to the south to intersect the predominant vertical joints. Holes were first drilled on 25-ft centers with intermediate holes on 12.5-ft centers. Closer spacing was necessary in some areas. The depth of grout penetration varied from El. 680 to El. 720, depending on the rock structure.

Drilling and Grouting Equipment and Materials

A maximum of sixteen diamond drills and six large-diameter calyx core drills were driven for foundation treatment in addition to several pneumatic wagon drill holes. Diamond drills were powered by four-cylinder 10-hp gasoline engines except when used in tunnel or galleries where an electric motor was substituted. Calyx core drills were driven by either 20-hp electric motors or 35-hp gasoline engines. Manufacturers' standard bortz bits were used for diamond drilling. Calyx core bits were 38 in. in diameter or smaller, and casings were made of $\frac{1}{4}$-in. steel from 54-in. to 33-in. diameters.

Seven grout mixers were used on the project, two of the type developed at Boulder Dam in Arizona and Nevada and five of the type developed at Norris Dam in Tennessee.[5] All grouting was done with 7-in. by 5-in. by 10-in., air-driven, double-acting, reciprocating pumps equipped with rubber valves and iron fluid pistons with replaceable rubber sleeves. The pumps and mixers were mounted together on skids so that they could be moved easily. The grout pumps were also used extensively for pumping water from tunnels during mining operations; and, during the treatment of the north core wall, these pumps were used successfully in a vertical position suspended in the calyx drill

[5] "Foundation Experiences, Tennessee Valley Authority: A Symposium," *Transactions*, ASCE, Vol 106, 1941, p. 687.

hole from an A-frame at the surface. In this position the pump did not require priming as the discharge line to the top of the hole remained full of water when the pump was not operating and provided a continual prime. This pump also operated satisfactorily when completely submerged in water.

The use of packers was limited at Fort Loudoun because of the stage-grouting methods generally required. In practically all instances where packers were used, the grout found its way through open channels and into the hole above the packers, thus grouting them in. For this reason, the use of the packers, in general, was abandoned.

For foundation grouting standard Portland cement mixed with water was used entirely, with the exception of the north reservoir rim and saddle dam as previously discussed. The water-cement ratio varied between 3.0 and 0.5 by volume. Grouting in an area was usually started with a thin grout and the water-cement ratio was gradually decreased. As pressures increased indicating possible refusal, the water-cement ratio was again increased.

Summary

This paper describes the treatment of solution channels and cavities in a limestone foundation at unprecedented depths below the water table. The conditions described indicate that structural deformational features rather than rock solubility often control the depth and extent of weathering. In the Fort Loudoun limestone foundation, solution occurred down to a depth of 300 ft below the original ground surface and 260 ft below the water table. The deep mining operations both for consolidation and an effective cutoff challenged the combined efforts of the designing engineers, field engineers, geologists, and special construction personnel, all of whom contributed to the satisfactory solution of this difficult and unusual foundation problem. Contrary to the normal conception that relatively more cavities are to be expected in the purer limestone, the cavities in this area were primarily controlled by solution along geologic structures—faults, joints, and bedding planes, rather than by the purity of the rock. On this project the geologists really came into their own and to them due credit is given.

DOUGLAS DAM

BY T. F. TAYLOR,[6] ASSOC. M. ASCE

SYNOPSIS

The foundation rock beneath the concrete structure of Douglas Dam and part of the adjacent rim was dolomitic limestone of the Knox formation. Large bedding cavities existed at each abutment of the concrete dam and the rock in the original river bed was highly cavernous. That part of the rim which was formed by shale was relatively tight.

The procedure used to prepare the foundation for the concrete dam consisted of excavating the highly cavernous rock, and mining, cleaning, and backfilling with concrete the large bedding cavities that were overlain with sound rock. These operations were supplemented with an intensive grouting program over the entire foundation for the concrete dam.

The reservoir rim adjacent to the dam was grouted principally where the outcropping rocks were limestones. Very little foundation treatment was needed in the shale rim. An analysis of final grout consumptions in all areas treated show that a satisfactory foundation was obtained. Observations made more than six months after the reservoir had been filled to maximum pool level indicated that seepage from the reservoir did not exist.

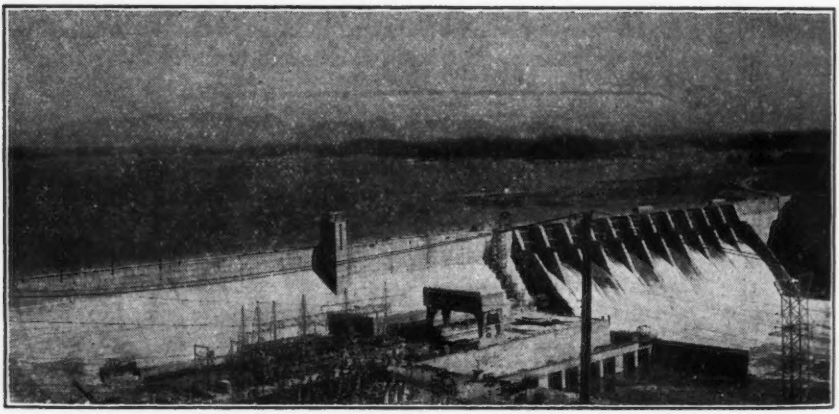


FIG. 14.—DOUGLAS DAM

INTRODUCTION

Douglas Dam is in Sevier County, Tennessee, on the French Broad River approximately 32 miles above its mouth (see Fig. 1). The site was located at

[6] Supt., Tennessee Eastman Corp., Oak Ridge Tenn.; formerly Constr. Engr. Douglas Dam, TVA Dandridge, Tenn.

a convergence in an otherwise broad valley. The original river channel at that point was about 500 ft wide at normal flow, with the bottom at approximately El. 868, and flanked on each side by a relatively narrow alluvial flood plain, on the average at El. 880. The south bank of the river was an almost vertical rock bluff. The north bank sloped away in the form of a steep sodded hill.

The concrete dam is approximately 200 ft high above the lowest point of the foundation and is 1,682 ft long at the crest. The main dam and powerhouse contain about 542,000 cu yd of concrete. Fig. 14 is a view of the completed dam, and Fig. 15 shows the plan of the structure and the geologic profile along the axis.

The topography of the reservoir rim, extending approximately 5 miles from the south end of the concrete dam, required the construction of eight earth dams in existing low saddles. The largest of these earth saddle dams contains 474,000 cu yd of compacted clay, and the total of all the earth saddle dams was 532,000 cu yd.

The foundation for the concrete dam is composed of dolomitic limestone of the Knox formation. The dolomite extends southward along the reservoir rim for a distance of 1,500 ft to Station 35+00. At this point, a relatively narrow band of limestone of the Lenoir formation begins and continues to a contact with the Athens shale on the south. The contact between the Lenoir limestone and the Athens shale is under the north end of the first and largest of the saddle dams. The remaining saddle dams are constructed on shale.

Preliminary exploratory borings at the dam site consisted of diamond core drill holes on three parallel ranges. The range lines were spaced 100 ft apart and were approximately parallel to the proposed axis of the dam. Holes were drilled on these ranges at 100-ft intervals, except at the south abutment where a few holes were drilled at 50-ft intervals. The holes penetrated from 100 ft to 250 ft into bedrock.

The exploratory drilling completed prior to the start of construction gave a general picture of the foundation, including the approximate elevation of sound, unweathered rock. It also indicated that, in the river bed, deep solution and weathering caused the formation of an extensive honeycomb of cavities and fissures. The weathering and solution apparently had not developed into any regular cavernous system.

In addition to the preliminary borings, considerable exploratory drilling was done during actual construction operations to develop accurate geologic sections. These sections were used for determining final excavation grades and for planning the general program of foundation treatment. Exploratory work was done with diamond core drills supplemented by a number of large-size, 42-in., shot drill holes to check questionable areas. The shot drill holes were planned to become a part of a tentative method of foundation treatment. A general section of the foundation along the axis of the dam is shown in Fig. 15(*b*).

Under the north abutment it was noticed that air was passing continuously in and out of some of the exploratory holes. These "blowing" holes indicated the presence of an extensive cavity in the foundation rock strata. It was

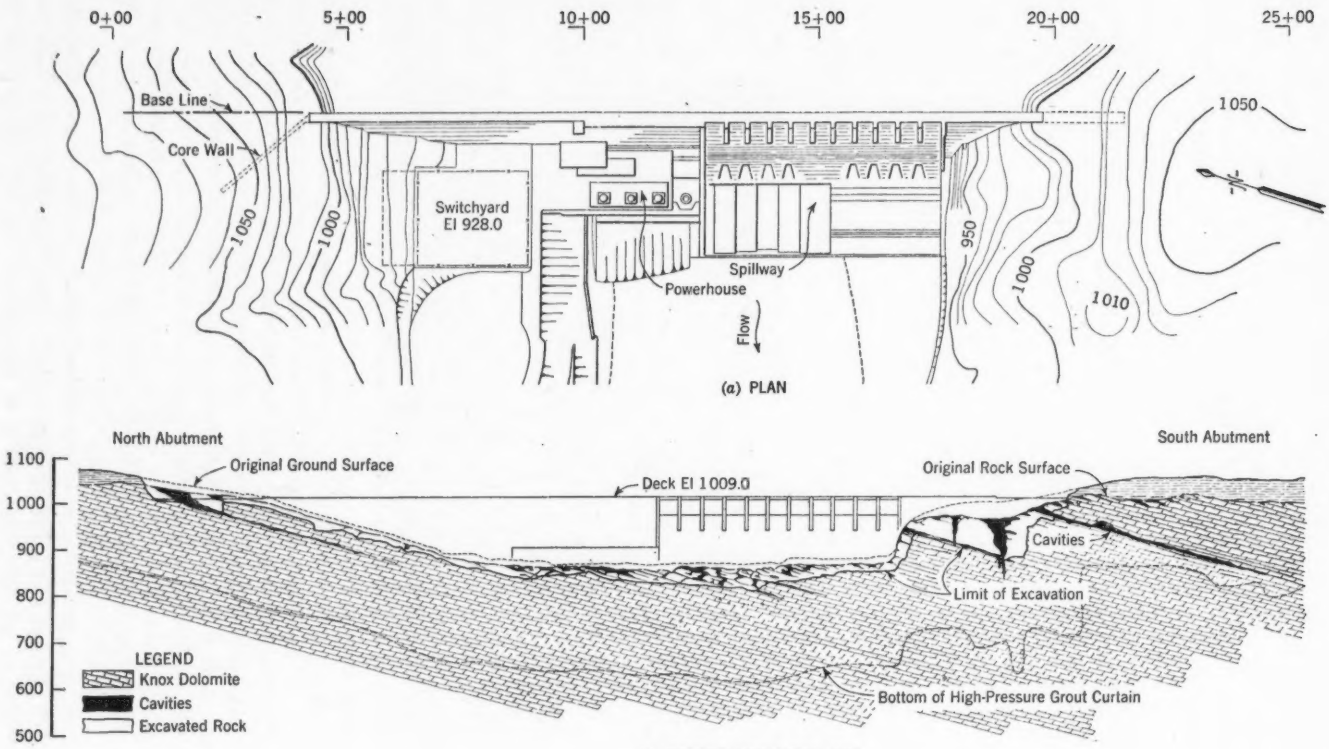


FIG. 15.—PLAN OF DAM AND GEOLOGICAL PROFILE

anticipated that this cave would have to be explored, and would receive special treatment.

At the south abutment a large bedding-joint cave was encountered which extended for some distance into the reservoir rim beyond the south end of the concrete dam. Drill holes in the rim indicated that no special foundation problem existed beyond the limits of the cave.

The anticipated program for foundation treatment consisted mainly of a general procedure for the concrete dam, plus a special treatment of the cavities existing at the north and south abutments and some curtain grouting in the reservoir rim areas.

North Abutment

The natural slope of the north riverbank at the dam site was nearly parallel to the dip of the foundation rock. Weathering in the upper strata of bedrock was fairly uniform and averaged from 8 ft deep to 10 ft deep over most of the

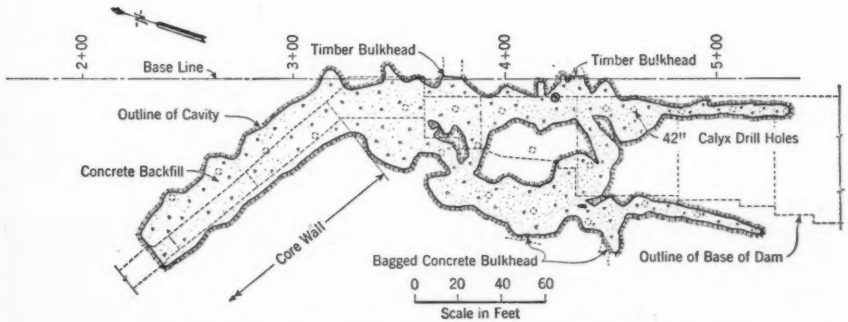


Fig. 16.—Bedding-Joint Cavity Under North Nonoverflow Section

area. The weathered rock was removed by open cut methods to expose good foundation rock.

The area in which the original exploratory drilling had indicated that an extensive cave developed along a bedding plane was further checked with 42-in. calyx drill holes. It was found that the cave extended under most of the area occuplied by the north nonoverflow section of the dam and also under the north core wall. The location of the cave is given in Fig. 16, with the outline of the base of the dam shown superimposed on the plan of the cavity.

The bedding-joint cave was partly filled with clay, and it was intercepted by several strike joint cavities running normal to the axis of the dam. These strike joint cavities formed enlargements in the main cave, which made the maximum height of the cave approximately 5½ ft. About 40 ft of sound rock existed above the roof of the cave after the foundation for the dam had been excavated to final grades. The removal of this sound rock above the cave was not considered practicable or desirable. To provide adequate bearing and watertightness, it was decided to mine out and fill with concrete that part of the cavity lying under the area of the concrete dam and core wall.

Fig. 15.—Plan of Dam and Geological Profile

To accomplish the mining and concreting operations, 42-in. calyx drill holes were drilled from rock surface to the roof of the cave on approximately 25-ft centers. Small hoists were set up over the drill holes to remove the muck and excavated material from the cave. After the cavity was mined out, it was thoroughly washed and cleaned with compressed air and water. Fig. 17 shows a section through the cave and gives the position of the large calyx drill holes.

Near the south end the height decreased so that it was necessary to enlarge the cave to provide working room for cleaning and washing. At the point where mining methods were stopped, the cave was approximately 0.6 ft thick.

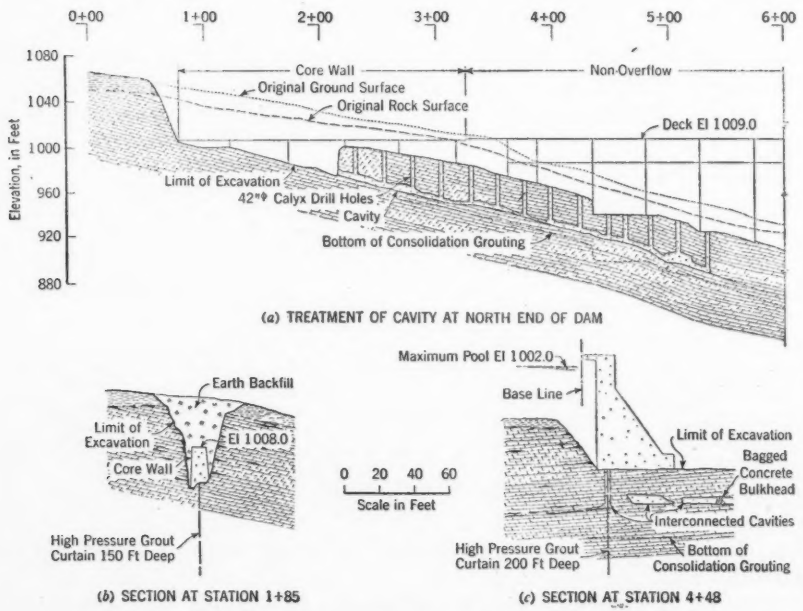


FIG. 17.—DETAILS OF TREATMENT FOR THE NORTH BEDDING CAVITY

As shown in Fig. 16, there were comparatively few solid rock pillars supporting the roof of the cave.

Large quantities of water were used in the washing operations; but it was not necessary to install pumps in the cavity. The cave extended upstream and downstream far enough so that the water drained freely by gravity.

At the north end of the core wall, the rock was extensively weathered above the roof of the cave. Therefore, it was necessary to excavate by open cut methods until the core wall could be extended to the floor of the cavity. This is shown in Fig. 17. In all cases it was found that the floor of the cave was formed by a thin shaly bed which retarded the weathering below it.

Formwork was necessary in some cases to confine the limits of the concrete backfill and to establish construction joints. The original 42-in. calyx drill

holes for mucking purposes were also used to place concrete. Filling was started from the calyx drill holes at the south end of the cavity and advanced northward. The floor of the cave received a thick bed of sand-cement mortar placed immediately in advance of the concrete—and as speedily as possible, to provide a hydrostatic head of fluid concrete. This method proved satisfactory; by filling the calyx drill holes with concrete, sufficient head was created to provide a relatively tight fill against the roof of the cave.

A total of 1,298 cu yd of concrete with $\frac{3}{4}$-in. (maximum size) aggregate was used to backfill the cave. The average mix had a water-cement ratio of 0.55 by weight and contained 1.83 bbl of cement, 0.79 tons of sand, and 0.74 tons of coarse aggregate per cubic yard of concrete.

After the cave had been completely backfilled, it was further treated by consolidation and curtain grouting with neat cement. The grout holes were extended through the cave from the foundation of the dam.

River Bed

The upper 20 ft to 60 ft of bedrock in the original river channel were extremely cavernous. This section formed the foundation for the spillway and a part of the powerhouse and intake section of the dam. Large cavities were found along many beds and were interconnected by cavities extending along both strike and dip joints. Some of the cavities were large enough to be termed caves and were continuous for long distances, principally along the strike of the rock. Solution was intensified because the strike of the rock was approxi-

Fig. 18.—Typical View of Rock Conditions in the River Bed Foundation Prior to Final Excavation

mately parallel to the flow of the river. The cavities were partly filled with clay, sand, and gravel, and some of the foundation was so greatly weathered that it presented the appearance of clay and gravel in which nested huge floating boulders. Fig. 18 shows this condition.

The excavation of this cavernous rock presented an extremely difficult construction problem because of the large flow of water coming into the coffer-

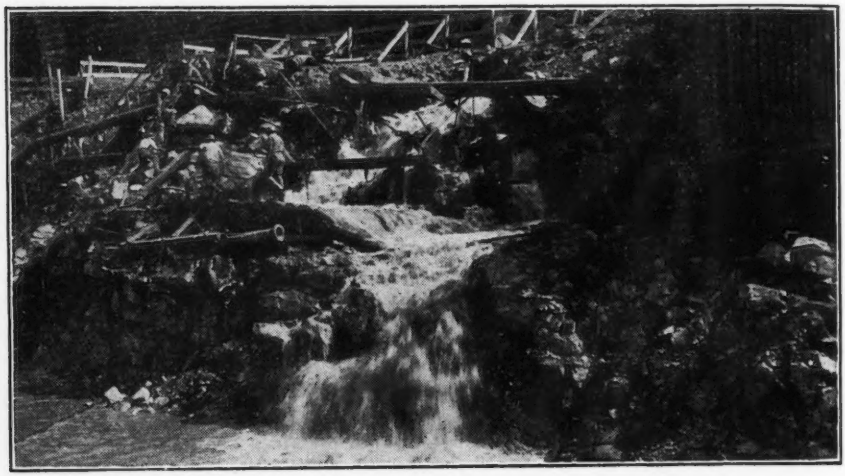


FIG. 19.—CAVERNOUS ROCK AND LEAKAGE UNDER THE UPSTREAM COFFERDAM

dam area from the various cavities. When the first-stage cofferdam was unwatered, large leaks or boils occurred in the river bed which threatened the entire excavation program.

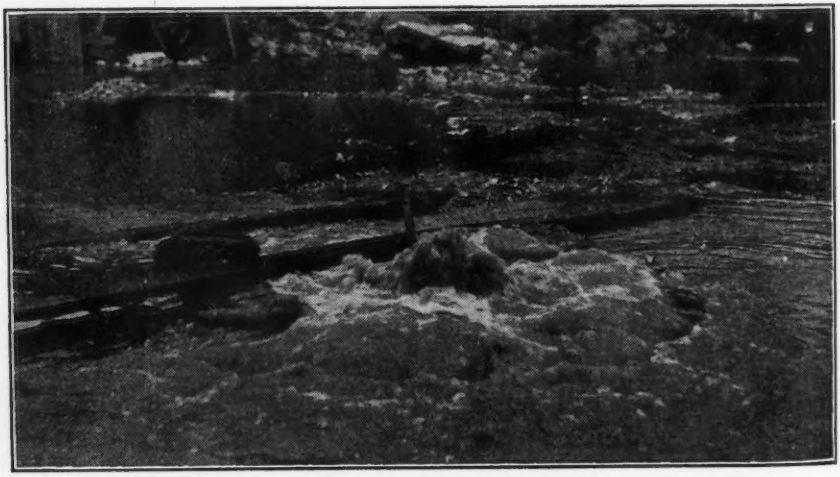


FIG. 20.—WATER BOIL IN THE EXCAVATION AREA

To continue excavation, extensive grouting with asphalt, clay, and cement mixtures and with other materials was tried in an effort to cut off the flow of water coming in the work area. This was supplemented by large-scale clay blanketing in the river bed both upstream and downstream from the cofferdam.

In asphalt grouting, care was exercised to prevent asphalt from entering the foundation rock lying under the area of the dam below the final excavation grades. Although these efforts succeeded in cutting off a large part of the inflow of water, leakage was still a major problem during excavation operations. For a considerable period, thirty pumps discharging 100,000 gal per min were in constant operation to keep the cofferdam area dewatered sufficiently to permit excavation to progress. Fig. 19 shows the cavernous rock beneath the upstream arm of the first-stage cofferdam. One of the many large leaks is shown and asphalt may be noticed where it had been used in cofferdam grouting. Fig. 20 shows one of the many boils of water in the middle of the excavation area. Although the construction difficulties and the cofferdam grouting are not part of the foundation treatment program, they are mentioned to indicate the scope of the foundation problem in the river bed.

As the weathered and cavernous rock was removed and excavation progressed in depth, the leakage was confined to the sides of the excavated area. Despite construction difficulties, all foundations were excavated to sound, principally unweathered rock; and, by isolating small areas, concrete for the dam was placed on a dry foundation. A few relatively minor seams remained below the final excavation grades. These were treated with consolidation and curtain grouting.

South Abutment

The alluvial flood plain on the south side of the river at the dam site ended at a vertical rock bluff approximately 75 ft high. Beyond the top of the bluff was a gently sloping upland. From the top of the rock bluff southward for a distance of 327 ft, the rock formed the foundation for the south nonoverflow section of the concrete dam. Early exploration indicated that extensive excavation would probably be required in this abutment. Prominent deeply weathered strike joint solution channels extended through the foundation normal to the axis of the dam. In addition to these solution channels, there were less pronounced bedding cavities. Terrace clay and gravel were present in the overburden as well as in the cavities.

The program followed in the south abutment area consisted of excavating by open cut methods to a well-defined bedding cavity. Below this cavity the vertical fissures were mined out and backfilled with concrete. Fig. 21 shows the excavation in the south abutment and one of the major strike joint solution channels. The area was excavated 134 ft below the original ground surface, 115 ft being rock.

The exploratory drilling also revealed the presence of a large bedding cavity beginning at the end of the south nonoverflow section of the concrete dam and extending into the rim. The cave was partly filled, and at some points tightly filled with clay, sand, and terrace gravel, with the terrace gravel normally in lenses. Exploratory drilling indicated the presence of the cave for approximately 700 ft between Stations 20+0 (El. 963) and 27+0 (El. 790).

Enlargements of the cave existed at strike joint solution cavities. The cave was excavated by drift mining supplemented by 42-in. shot drill holes on 50-ft centers extending vertically from the ground surface to the floor of the cave. A drift was excavated following the cavity down the dip from a portal

FIG. 21.—STRIKE JOINT SOLUTION CHANNEL IN SOUTH ABUTMENT

near the end of the concrete dam. Fig. 22 shows a section through the cave and gives the position of the shot drill holes. Mining operations were extended for a distance of 410 ft from El. 963 at Station 20+0 to El. 860 at Station 24+10. The height of the cavity ranged from 3 ft to 10 ft except for one section 50 ft long, which was 25 ft high. In some instances, it was necessary to excavate rock to obtain sufficient working room in the cavity. During the mining operations no difficulty was experienced with ground water until the cavity reached El. 873, which was river level. Below this elevation, periodic pumping was necessary to allow excavation to proceed.

To prevent leakage, a concrete cutoff wall or plug was designed and placed in the cavity as a water stop. The concrete plug extended from the floor to the roof of the cavity. The cavity was cleaned and the concrete was placed by using the large-diameter shot drill holes. Concrete filling was begun at the low end of the cave and proceeded up the dip; and wood forms were used to maintain construction joints and the proper width of the plug. Care was exercised at the south end of the plug to obtain a solid tie into rock, at which point the plug was widened considerably. The plug contained 2,458 cu yd of concrete using a maximum size of aggregate of 1½ in.

After the concrete had contracted, the joint between the roof of the cave and the top of the concrete plug was grouted. Where necessary, openings be-

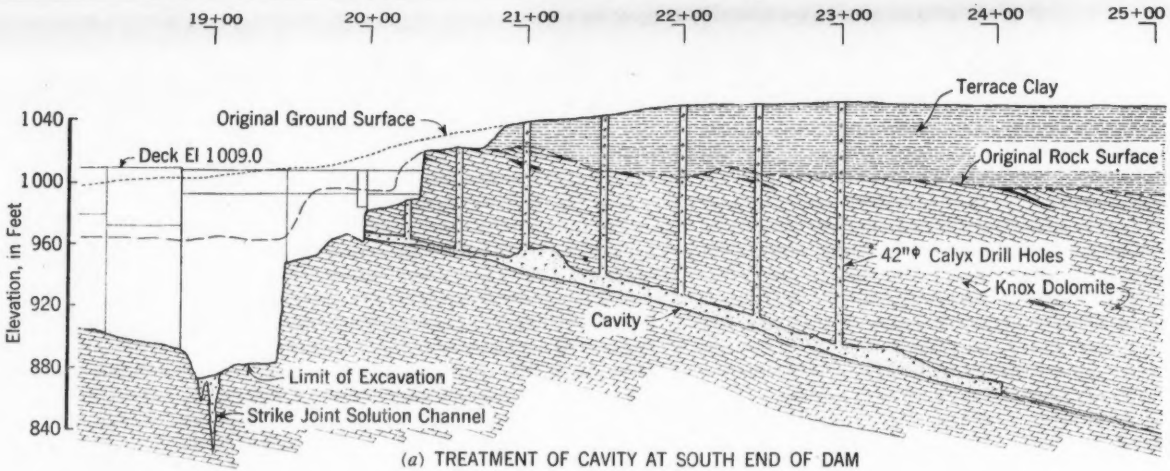


FIG 22.—DETAILS OF BEDDING CAVITY AT THE SOUTH ABUTMENT

tween the roof of the cavity and the concrete plug were calked to prevent grout leaks, and the grout was supplied through nipples previously placed on approximately 20-ft centers along the roof of the cavity. The detail of these grout outlets and of the concrete plug is shown in Fig. 22. As an added precaution, clay was placed to fill the cavity both upstream and downstream from the concrete plug.

An extensive program of cement grouting followed the completion of the concrete plug, with the drill holes extending from the ground surface through the concrete filled seam. This procedure made the concrete plug a part of the grout curtain extending from the dam into the rim area.

South Reservoir Rim

Beyond the south end of the concrete dam, the rim is formed by Knox dolomite for a distance of 0.4 mile. A limestone of the Lenoir formation begins at this point and continues for a short distance to a contact with the shale of the Athens and Sevier formations. The shale ridge extends southward with an average width of 1,000 ft, at a maximum reservoir elevation of 1002. The highest point of the ridge is approximately at El. 1240, and the shale ridge drops below a design crest of El. 1019 at seven saddles. At each of these saddles an earth dam was required in addition to a large saddle dam at the contact of the limestone and shale.

The part of the rim underlain by the Knox dolomite is covered by a blanket of terrace clay and gravel which averages 43 ft in thickness. Most of the Lenoir limestone rock comes to the surface, and there are many exposed ledges and pinnacles of limestone.

Except for the one cave at the end of the concrete dam, there was little evidence of solution and weathering in the rocks of the south rim. Weathering had not affected the Athens shale except for a few minor beds. It was found that the unweathered shale was not absolutely impervious as core drilling encountered streams of water at relatively low levels. The beds in the Sevier shale were folded and faulted. The shales were somewhat calcareous and contained a few interbedded layers of shaly limestone. The entire formation was essentially insoluble and without cavities or deep weathering.

North Reservoir Rim

The alluvial flood plain on the north bank of the river at the dam site ends in a steeply sloped sodded bank which rises to El. 1200 approximately 0.3 mile from the north end of the concrete dam. The underlying rock is the Knox dolomite formation.

Preliminary borings in the short section of the north rim revealed well-defined cavities which in general followed bedding planes. This drilling showed the water table at a relatively low elevation—indicating the probability of open cavernous rock. The north rim was treated with an extensive high-pressure grout curtain to prevent leakage around the end of the dam.

Grouting of the Concrete Dam Foundation

Consolidation Grouting.—The entire foundation beneath the concrete dam and powerhouse, with the exception of the spillway apron, was consolidated

with neat cement grout for a depth of approximately 40 ft. The drilling and grouting progressed in stages with the holes on a regular pattern. The first-stage holes were placed on 20-ft centers parallel and perpendicular to the base line of the dam. Second-stage holes were placed midway between first-stage holes, third-stage holes midway between second-stage holes, etc. The drilling and grouting of a stage in an isolated area were completed before the next stage was begun. The quantity of grout consumption in a hole in one stage determined whether an additional stage hole was required. The strict theoretical pattern of holes was varied in some places. In general, wagon drills were used for all the consolidation holes. In some cases where the total depth of hole exceeded 40 ft, diamond core drills were used. All holes were vertical except that, in places where known vertical solution channels existed, some angle drilling was done to obtain most effective grouting.

The consolidation holes and the seams intercepted by them were washed with air and water under pressure through nipples grouted into the drill holes at the foundation surface. No attempt was made to wash or to grout individual seams in the consolidation holes. The holes, however, were water tested to determine the proper consistency of neat cement grout with which to start the grouting treatment.

The area that formed the foundation for the north nonoverflow section of the dam was given special treatment because of the cavernous area beneath. The consolidation holes were extended through the concrete backfilled cave, and every effort was made to secure a tight seal between the top of the concrete and the roof of the cave.

When consolidation grouting was first begun in the north abutment it was noticed that the grout traveled several hundred feet downstream from the limits of the dam. To prevent the grout from traveling into areas where it was not needed, the limits of the foundation of the dam were outlined with drill holes 40 ft deep spaced on 5-ft centers. These holes were grouted with a thick cement grout fed by gravity to form a curtain to confine the consolidation grout. This method probably eliminated a large percentage of grout waste.

For all consolidation grouting, a maximum pressure of 40 lb per sq in., measured at the surface of the hole, was used. In general, most of the grouting was completed prior to placing concrete in the base pours of the dam. Uplift gages were installed at intervals for checking any possible uplift in the foundation rock. The grout used was composed of neat cement and water and varied from 0.6 to 2.0 parts of water to one part of Portland cement by volume.

Low-Pressure Curtain Grouting.—A grout curtain to form a water cutoff in the foundation beneath the dam was extended in a line just downstream from, and parallel to, the upstream face of the concrete dam. This curtain was formed by grouting shallow holes, in general averaging about 50 ft in depth. Pressures not exceeding 40 lb per sq in. were used. The low-pressure curtain was drilled and grouted in conjunction with the consolidation grouting. The treatment given these holes was similar to the consolidation program, except that the low-pressure curtain holes were treated more intensively and were drilled mainly with diamond core drills.

Drilling and grouting were done in stages with the first-stage holes on 24-ft centers; the second-stage holes reduced the spacing to 12 ft; the third-stage holes reduced the spacing to 6 ft; etc. The resulting minimum spacing at any one place on the low-pressure curtain was 1.5 ft; and the maximum spacing, 12 ft. Water testing was used to determine the initial grout consistency and grout mixes and pressures were identical to those used in consolidation grouting.

High-Pressure Grout Curtain.—The relatively low elevation of the water table at the dam site, together with known information regarding seams and cavities, indicated the need of a deep high-pressure grout curtain along the upstream face of the dam. To determine the depth of this curtain, a number of deep check holes were drilled, water tested, and grouted. The results obtained in the check drilling and grouting of these holes indicated that the curtain should extend to a maximum depth of 200 ft. A number of small seams were encountered between 100 ft and 200 ft below the surface of the foundation.

The high-pressure curtain followed the line of the low-pressure grout curtain. Fig. 15(*b*) shows the relative position of the grout curtain. The drilling and grouting were done in stages with the first-stage holes placed on 18-ft centers. Successive stages reduced the hole spacings by one half, and the number of stages required for an area was determined by the grout consumption of the preceding holes. The minimum spacing of holes at any one point in the final curtain was 1.4 ft; and the maximum spacing, 9 ft. The depth of drilling was decreased where the grout consumption of previously drilled holes indicated a satisfactory curtain at lower levels. The minimum depth of holes was maintained at 100 ft.

The normal procedure was to wash and grout the lowest known seam or cavity. The packers were then raised and the operation repeated with each seam until all had been grouted. This procedure allowed each seam to be washed and grouted prior to the beginning of the washing of the overlying seam. Ample time was allowed between grouting and washing operations to permit the grout to harden. This process was accomplished by providing sufficient flexibility in plant equipment to allow either washing or grouting to be conducted at other locations.

Almost all the drilling and grouting were done from the lower drainage gallery after the bulk of the concrete had been placed. Thus, additional protection was afforded against uplift when using the higher grouting pressures. The maximum grouting pressure was 200 lb per sq in. measured at the top of the hole, and was in excess of the hydrostatic pressure on any seam, but less than the unit weight of the overlying rock and concrete. A neat cement grout, varying from 0.6 to 5.0 parts of water to 1 part of cement by volume, was used.

The results of the high-pressure curtain grouting as accomplished by successive stages are illustrated in Fig. 23(*a*). This curve shows the grout consumption per foot of depth of hole for each successive stage of grouting.

Rim Grouting

South Rim Grout Curtain.—A grout curtain was provided extending from the south end of the concrete dam along the reservoir rim. Tied into the end of the high-pressure grout curtain beneath the concrete dam, this curtain ex-

tended to the south end of the first saddle dam. Extensive drilling and grouting operations were employed in the area between Station 20+50 and Station 27+00 to check, tighten, and provide a water stop around the south end of the concrete dam. These operations also completely checked the cave that had been mined out and backfilled with a concrete plug.

The drilling and grouting were done in stages—as many as were necessary to check any area of high grout consumption—with the first-stage holes on 100-ft centers. In some cases, five stages were used, resulting in hole spacing of 6.25 ft. All holes were driven with diamond core drills to El. 860 or lower—slightly lower than the elevation of the original river bed.

No seam washing was attempted in the rim grouting. Neat cement grout with calcium chloride added to accelerate the setting time was used.

The neat cement grout varied from 0.6 to 2.0 parts of water to 1 part of Portland cement by volume, with 3% of calcium chloride ($CaCl_2$) by weight of cement added. The effect of the rim tightening by successive grouting stages is indicated in Fig. 23(*b*), which shows the grout consumption per foot of depth of hole for each stage.

Check drilling and grouting indicated that no other intensive grouting program was necessary in other parts of the south rim except at saddle dam No. 1, where considerable grout was injected, particularly at the contact between the Lenoir lime-

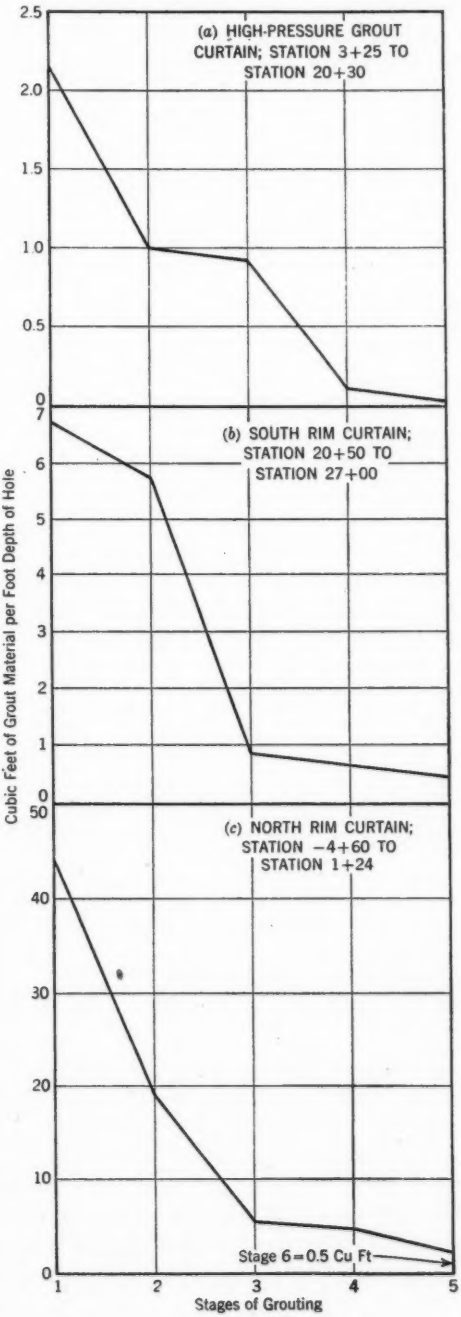

(a) HIGH-PRESSURE GROUT CURTAIN; STATION 3+25 TO STATION 20+30
(b) SOUTH RIM CURTAIN; STATION 20+50 TO STATION 27+00
(c) NORTH RIM CURTAIN; STATION −4+60 TO STATION 1+24
Cubic Feet of Grout Material per Foot Depth of Hole
Stage 6=0.5 Cu Ft
Stages of Grouting

FIG. 23.—RATE OF GROUTING CONSUMPTION

stone and the Athens shale. The drilling and grouting under this saddle dam followed the same procedure used in the rim area south of the concrete dam.

North Rim Grout Curtain.—The north reservoir rim possessed some natural protection against seepage in the form of a long earth blanket. However, the number of well-defined dip cavities lying along the line of the grout curtain predicted a heavy grout consumption. It was considered neither necessary nor feasible to mine out and backfill these cavities with concrete; but it was felt that a clay grout or a grout containing clay in a prescription mixture with cement would provide a satisfactory cutoff against seepage. A clay and cement mixture in the proportion of 1 part cement to 4½ parts of clay and 6 parts of water by volume was tried. The grout was pumped into the drill holes as a slurry and large quantities were consumed. The maximum consumption per hole was 20,905 cu ft, even though the pressures used were only slightly above gravity and the consumption was limited to 500 cu ft before suspending grouting operations and allowing the mixtures to set for a period of three days. It was found that the clay mixture was spreading much farther than was necessary for the formation of a grout curtain; and, accordingly, the clay-cement mixture was abandoned in favor of neat cement grout with calcium chloride ($CaCl_2$) added to accelerate the setting time.

The drilling and grouting were done in stages with the first-stage holes on 100-ft centers. Six successive stages were required, which reduced the hole spacing to 3.12 ft. The neat cement grout varied from 0.6 to 2.0 parts of water to 1 part of Portland cement by volume with 3% of calcium chloride ($CaCl_2$) by weight of cement added. The results of the north rim tightening are shown in Fig. 23(*c*), which gives a curve of the grout consumption in cubic feet per foot of depth of hole for each successive stage to stage five. At stage six (not shown) the grout consumption had decreased to 0.5 cu ft.

TABLE 3.—Drilling and Grouting Quantities, Douglas Dam

Section	Structure	Drilling Quantities (Lin Ft)			Grout (cu ft)
		Wagon[a]	Diamond[a]	Total	
North rim	Curtain	...	15,758	15,758	145,919
Concrete dam[b]	Consolidation	51,700	16,714	68,414	33,806
Concrete dam[b]	Low-pressure curtain	8,160	13,164	21,324	9,702
Concrete dam[b]	High-pressure curtain	...	39,380	39,380	51,340
South rim	Curtain	...	23,991	23,991	76,669
Total	.....	59,860	109,007	168,867	317,436

[a] Wagon drill holes, 2½ in. in diameter; and diamond drill holes 3½ in. in diameter. [b] Only neat cement grout was used in the concrete dam.

Summary and Comparison of Drilling and Grouting Quantities

The task of conducting the foundation treatment program in conjunction with a rapid construction schedule was difficult. A judicious disposition of men, materials, and equipment was necessary to obtain satisfactory results and

costs. Approximately three hundred men were utilized on the drilling and grouting program. Table 3 shows the linear feet of drilling for the various features and the quantities are separated to show wagon drilling and diamond drilling as well as the quantities of grout consumption. An interesting comparison of the results of the grouting may be obtained from the three curves in Fig. 23.

The rock conditions were different for each case as demonstrated by the rate of grout consumption per foot of depth of hole in stage one of grouting. The north rim having large cavities took 45 cu ft per foot of depth; and the concrete dam where rock was good, and also preconsolidated, took only 2.2 cu ft. On the south rim where the rock was reasonably tight and a concrete plug was constructed, the first-stage rate was 6.7 cu ft per foot of depth. A satisfactory tightening was obtained by the use of successive stages of grouting and by intensive treatment. The rate of grout consumption in the final stage was 1 cu ft or less per foot of depth of hole in all cases.

Drilling and Grouting Equipment

Standard equipment was used for drilling and grouting. Two-compartment, mechanically-agitated, mixers were used, and the grout was pumped by 7 by 5 air-driven duplex reciprocating pumps. Standard manufacturers' set bortz bits were used on the diamond core drills and the core drills were both air driven and gasoline motor driven. Two types of packers were used in the grout holes—a simple mechanical packer and an expanding rubber packer.[1]

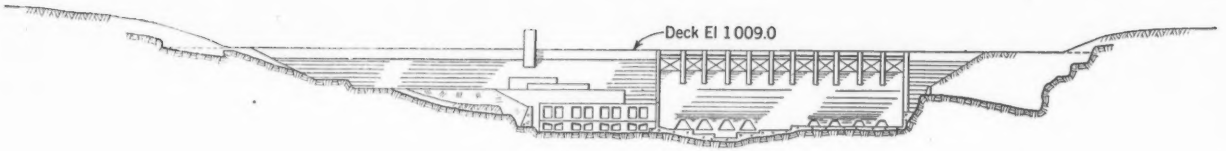
During peak operations on the job, thirty wagon drills, fourteen diamond drills, eight grout mixers, and three large-diameter shot drills were employed.

Ground-Water Investigations and Foundation Drainage

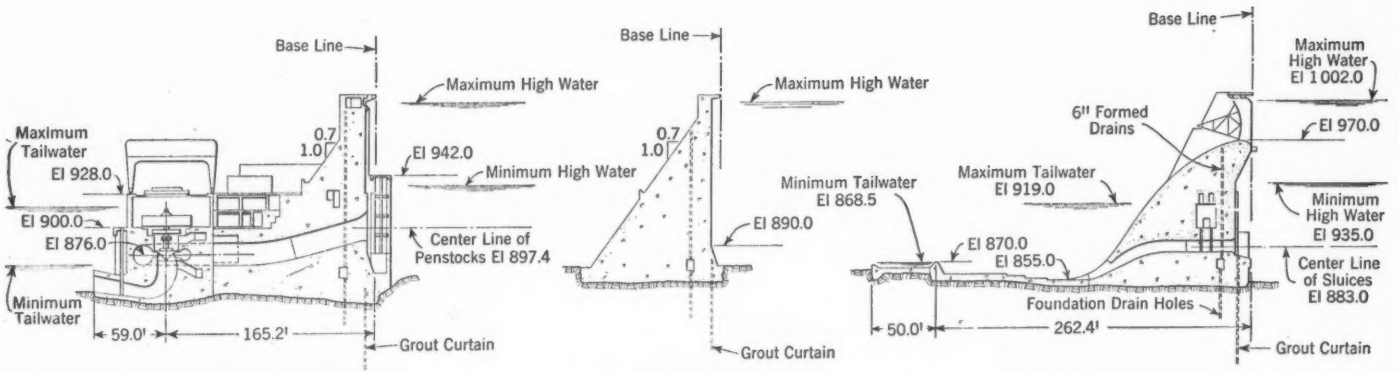
Ground-Water Investigations.—The areas adjacent to the concrete dam and strategic points in the reservoir rim were watched carefully for seepage during the period of reservoir filling. A program of ground-water wells was laid out, and wells were drilled for ground-water measurements. All holes were driven below river bed to approximately El. 868.

Observations were made of these wells regularly and also of any additional existing farm wells. Hydrographs of the ground-water table were plotted. A gradual rise and adjustment in the water table was noted as the reservoir rose, but in no case was there any drastic change or any sign that would indicate leakage from the reservoir.

Foundation Drainage.—Foundation relief drains were drilled beneath the concrete dam just downstream from the high-pressure grout curtain. Fig. 24 shows the position of the drain holes. These holes were drilled 30 ft deep into foundation rock and were placed on 8-ft centers. The relief of the holes was in the lower drainage gallery. All the foundation drain holes were drilled after the foundation grouting program was completed. When the reservoir had filled to within 2 ft of the maximum pool level, the foundation drains had a flow of only 56 gal per min.



(a) DOWNSTREAM ELEVATION



(b) POWERHOUSE AND INTAKE SECTION (c) NONOVERFLOW SECTION (d) SPILLWAY SECTION

FIG. 24.—SECTIONS SHOWING POSITION OF RELIEF DRAINS

A regular pattern of holes was also drilled over the area under the spillway apron. These holes were drilled 10 ft into foundation rock and extended through the concrete slab of the apron to provide relief from uplift pressures

Summary

Sufficient geologic information made possible advance planning and scheduling. This information was augmented by additional exploration during the construction period. Careful coordination of excavation and concreting with foundation treatment was essential, because of an extremely fast construction program. Substituting concrete for grout wherever feasible, as well as confining grout mixtures within required limits, minimized the waste of cement. Large-diameter core holes used in connection with mining and placing concrete plugs allowed visual subsurface examination and assured a positive cutoff.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

REPORTS

STEWARDSHIP OF PROFESSIONALISM GIVEN HIGH PRIORITY

ADDRESS AT THE ANNUAL CONVENTION, SPOKANE, WASH., JULY 17, 1946

BY W. W. HORNER,[1] PRESIDENT, ASCE

This is our first Summer Convention in three years. During those years we, together with all other Americans, have been concentrating on the war effort. A great many of our members have been in the armed services, and many others were engaged in activities essential to the successful and early conclusion of the conflict. It is a source of deep satisfaction to know that members of our Society, together with other men of science, played an important rôle in this most scientific of all wars. I am confident our pride in our part of the atomic bomb project, which contributed so much to hastening the cessation of hostilities, will be judged a pardonable one.

Now it is time to face the problem of peace with the same singleness of purpose and in the same spirit of unity which characterized our war effort. I am certain that the members of our Society are not unmindful of the innumerable problems which are vexing and perplexing the country in its uphill reconversion efforts, and although I could dwell at great length on what stake we, as engineers, have in the prompt and proper solutions of those problems, I shall not do so.

For this, it seems to me, is an occasion for discussion of Society affairs and of Society aims and efforts. Those of you who have been members of your Section membership committees at one time or another, and who have endeavored to explain to prospective members, the advantages of belonging to ASCE, will readily understand what I mean when I say, and heartily concur in my opinion that, the most intangible, though perhaps the most valuable, of all our aims is the one we call "professional improvements" of our members.

In choosing that phase of our activities for discussion here today I take particular cognizance of the increasing attention our Society has been devoting to the subject of its younger members, and, with the permission of you older

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NOTE.—This address is published for the record, and is not open to discussion.

[1] Cons. Engr. (Horner & Shifrin), St. Louis, Mo.

members, I should like to direct my ensuing remarks principally to my junior colleagues.

Not long ago, there came my way an opportunity for a man-to-man talk with members of a Student Chapter of the Society. It was a heart-warming and revealing experience, and one which stimulated my thinking on the subject of the younger members of our Society.

Most of these fine young men had returned from service in the armed forces to resume their war-interrupted education. Many had seen action in foreign lands. They were, I am happy to report, much more maturely serious than students I had known previously, and were keenly interested in discussing their future. They had definite ideas as to what they felt they ought to do.

I asked them if, in the light of recent events and developments, they had felt at all inclined to reconsider their decision to enter the engineering profession. Had they, I inquired, ever thought that there might be a greater reward in a craft under the protection of organized labor?

The replies I received were most definite. Yes, there had been a good deal of serious thinking devoted to the all-important subject of their future, and the glittering and immediate beneficial results held out to them by organized labor had received the attention such seeming prizes would naturally attract, especially from youth. But their decision to become members of a profession, rather than of a trade or craft, was declared emphatically. Definitely, they were in for the longer and harder pull. They expected to be free to develop their careers along constructive lines, and could not see the leveling process of union rules, which they felt would restrict their activity, as fitting into their picture of what they wanted for themselves in an America which they had fought to keep free. Theirs was no philosophy of doubt or compromise, in which "higher wages now" were to compensate for inhibition of initiative. They were concerned with fundamentals— opportunities for the future, salaries commensurate with professional status, and a satisfactory scale of living to reward them for their years of study, learning, and development into the careers of their choice.

And they were equally forthright in their inquiries as to what assistance they could look for from the older and established members of the engineering profession. What had the profession done to help them? What was it doing to assure them a fair chance? What were its plans for future aid to the younger fellows?

It was obvious that they weren't too well satisfied with what they had heard and that there was grave doubt in their minds that we elders had done all that it was possible for us to do. They inferred that possibly we had been sticking too close to our personal knitting and, by implication, asked whether we had not failed to develop a professional feeling to a reasonable degree.

Under the cross-examination of these straight-thinking young men, my derelictions were beginning to appear in the record, and I found myself feeling apologetic and definitely on the defensive. It became apparent that I, and not they, needed to reflect and reconsider.

I have done this at some length and have concluded, for myself and my contemporaries in the profession, that perhaps it is best that we plead guilty and ask the clemency of the new generation, who are so sure of themselves and so ready and willing to take over.

In throwing ourselves on the mercy of the court, so to speak, I think it just, however, that we be permitted to offer an explanation—hardly an excuse—for our conduct of our professional affairs to date. I think a review of such extenuating and mitigating circumstances as may be found to exist is in order at such a time, and I think, also, that I can best present it in terms of my own experience. This I shall proceed to do, presenting it (again with the leave of my older colleagues) in the form of remarks directed at our Junior Members and to the mature and earnest Student Chapter Members who will soon join them in our professional ranks.

When I entered the Junior grade of membership in ASCE, America was emerging from the gay nineties and plunging into the ensuing railroad panic of 1907. Immigration was still at high tide. Labor was plentiful and practically unorganized. Business and industry were enjoying freedoms they long since have lost—were, in fact, abusing their privileges in a manner so ruthless that it could lead nowhere but to the curtailment and abrogation of those privileges. It is significant that the loss of those freedoms and privileges, upon which we now look back as something which occurred overnight, was not, in fact, any single event which took place in anything even approaching so short a time. Deprivation of privilege came about gradually, as the people grew increasingly incensed over the abuses. Corrections were instituted piecemeal, as has been the case with remedial measures throughout history. Controls were instituted not only as the need for them arose, but in most instances only after the abuses were so flagrant as to cause the people to wonder if the means of abolishing them were to be forthcoming in time to avert national calamity. In short, just as today we are inclined to despair over abuses at the pendulum's other extreme, so did the people of the time of my youth fret and worry and fume and wrangle over the injustices and the inequities which menaced the opportunities which they sought and which they deemed it their right to have.

In those days, government was law and order and taxes, and had not yet entered business to an appreciable degree. Public works were largely municipal utilities and federal postoffices, and the taxes for their construction were so relatively slight that the average man did not have to concern himself about them. Certainly, a newcomer to the professional ranks of our Society, starting out to earn his livelihood as an engineer, was immune from them. Comparatively small though it was, his paycheck was all his own when he received it, and he was not afflicted with that relatively new malady, the "deducts," as are we today.

The American Society of Civil Engineers then was to the young man an order of eminent mystics who met primarily to compare notes on great achievements. Then the Junior was received into the Society after careful scrutiny and was thereafter seen, but for some time, not heard. Then, as now, the Junior needed a friend and friendly help, but the devil took the hindmost.

We Juniors did have impressed upon us sound ideas of professional integrity and an understanding that the results of our work must be always in the public interest. But the notion that we should devote some time to social and economic improvement of the membership was not advanced in any manner whatsoever. One got a job and, if fortunate, a chance to work under an accomplished elder. By hard work, he could rise in his profession. Individual engineers carried through classic projects and made great contributions to technological advance. Many members of the Society rendered conspicuous public service, and the profession profited and grew in the public esteem.

From World War I, we emerged with an increased prestige, for even the crude tanks and the handful of airplanes of that conflict, captured the public imagination to the extent that many called it "an engineers' war." From the war, we gained a broader professional solidarity, but our energy soon became diffused through a number of specialized societies. The ASCE began to modify its objectives in the direction of professional welfare with a faint sense of the beginning of the social revolution which we since have been undergoing. Technically and economically, we began to become closely related to industry, business, and public works, and we increasingly assumed major responsibility for technological advancement and for management and direction, as well as for our earlier endeavors in the realm of planning.

It is evident that we had little training in the service of the profession. The important social and economic side was at best a spare-time sideline for comparatively few members of the Society. Wiser heads recognized the need for enhancing the status of the profession, and occasionally voices sought to be heard on the subject. But the membership was busy, and the voices were heard but briefly and ineffectually. We were immersed so deeply in our individual improvement problems that for a long time we did not recognize the benefits or the essentiality of improving our position as a profession.

When it did come, our awakening was a gradual process. We created the Engineers' Council for Professional Development and supported it after a fashion. We saw the light on the need for legal recognition of the profession in the public interest, and got together to put through state registration. Our committee work began to point out somewhat more strongly the need for such things as professional recognition and adequate salaries and fees.

We were not well prepared to meet the bulge in the revolutionary tide that accompanied the economic depression and the New Deal. In that era, we did perform important service and the profession suffered comparatively little. While we had some economic distress of our own, we did not organize to produce widespread leadership, for which there was then great opportunity. A better conception of our responsibilities and a strong presentation of engineering ideals and ideas might have damped the swing to the left somewhat and might have placed your younger generation in a stronger starting position for the work you have to do.

In summary, we gave ourselves wholeheartedly to our engineering work; we made major contributions to developing the United States into the greatest and most capable nation of the world, a condition which was to stand her, and

us, in good stead when World War II required of us that we be the arsenal of democracy. So we had little time to organize ourselves as a profession. Yet some members—a small part of our Society, and all too few—worked in that direction. In the past decade particularly some orderly procedure has been developing. The membership of ASCE and of the other major engineering societies has been increasing rapidly, and a greater part of those memberships has displayed a growing interest in professional welfare.

The need for unity in the profession as a whole has resulted in the federation of Society Sections with other engineering bodies in the metropolitan areas and in some of the states. Most recently, Engineers Joint Council has met at least in part, the need for unified action on the national level and has proved to its five constituent societies that they have much in common, can work together in harmony, and in doing so can render public service of a higher order than has heretofore been possible.

World War II has left us with a still greater recommendation of our capacity for service. Out of the war has come the realization by the public of the value of research and technological development. It took the atom bomb to do it, but the country now looks to scientists and engineers to make secure the future of the nation, as well as to enhance its mode of living. Engineers are directly concerned with scientific research. Basic and applied research in engineering provide the tools with which we practicing engineers earn our livelihoods. We bear the same relationship to fundamental research in the science of engineering as does the practicing physician to the basic investigations of the medical scientist in medical and biological fields. It rests with us to make use of the fruit of scientific engineering research and to turn it to the practical service of our fellowmen. It is our duty to accept the challenge offered by the scientific developments of the war and to strive for the same degree of progress in peace as we made during the war.

But thorns, as well as roses, adorn our postwar path. Collective bargaining on the one hand and a growing regimentation of the men in public service into "bigger," if not always "better," bureaucratic agencies on the other, have focused the spotlight of self-searching analysis on our present weaknesses.

We have in our Society, and with our affiliations, created the skeleton of good professional organization. At the present time, that organization is undermanned; too few of us are actively engaged in the relatively new task of molding firm flesh onto that skeleton. Too many of the older members retain their earlier individualistic tendencies, and too many of all ages are too busy with private affairs, thus bidding fair to repeat the errors of our earlier ways to which I am now, as I stated, pleading guilty.

That is why I am especially pleased to see you younger men come into the picture. Your awareness of the social and political scene, your concentration on the intrinsic value of the professional attitude, your determination to see that matters you consider essential are acted upon, and the natural zest of youth which you bring with you, provide the new light and the new energy which should transform our slower progress into accelerated achievement.

Today, the organization of the American Society of Civil Engineers is well distributed over the country. Its foundations and its strength are in its Local

Sections. Your real service to the profession must necessarily be applied there and at that level it can be adjusted best to the circumstances of your job and your environment. As you come into the Local Section membership, you will invariably find that these member groups are fulfilling a first essential task, that of extending the acquaintanceship between professional men into friendships and sympathetic understanding of the other fellow's problems and performance. Many of the members quickly will find time to become interested in you and to be helpful as advisers. In some of the Sections, you will find an arranged elder brother system, which assures you of welcome, personal guidance, and opportunity for service in the directions indicated by your interests and talents. You will find generally that your Section has both social and technical programs, and that you can participate in them and in arranging for them to good advantage. If, in some instances, they have not been well organized, there is a constructive service waiting for you.

Many of the Sections are member bodies of local engineering councils or federations generally interested in public questions and in the development of intelligent engineering opinion and expression with respect to them. Volunteer to serve on their special committees. You will come to recognize the varying viewpoints of the different branches of the profession and how they may be reconciled. You also will be introduced to local civic and political problems, and will find an opportunity for direct public service—a field in which we engineers have been all too inactive.

Either in your Section programs or through your personal work you may find encouragement to enter civic or business organizations, with the further opportunity of broadening your viewpoint while acting constructively for the public. Separately, there is opportunity here to demonstrate to the citizenry at large that the engineer has more than a one-track, technical mind. To those who may be inclined to question the fitness of professional men injecting themselves into community affairs, rather than waiting to be consulted, I would point out that our professional brothers, the doctors and lawyers, have followed this pattern, not only have they avoided impairment of their professional status, but they actually have enhanced their community positions by manifesting an interest in nonprofessional affairs.

Should you find that your job brings you into the sphere of employer-labor relations discussions that may involve you in collective bargaining procedure, your Section officers can obtain from the Executive Secretary pertinent, up-to-date information as to what you had best do to protect your professional status. The Society's past activities on this score are too well known to you, I am certain, to require repetition here. What I do want to pause long enough to do, however, is to give reassurance to any who may need it that the Society's position on this all-important matter is unchanged and that we propose, within the restricting limitations which the laws of the country impose on us, to continue our efforts to insure that professional employees shall be guaranteed the right to determine directly by their own votes whether or not they wish to bargain collectively; and, where they desire, to do so through representatives of their own choosing in a bargaining unit of their own choice. We shall continue to oppose regimentation of professional men into heterogeneous labor

organizations in which they are outnumbered by nonprofessional men and where they have no mutuality of interests with the majority of the members.

In the past, Local Section activity has not been closely coordinated with matters of interest to the membership in the national field or of national legislation having a possible effect on the membership of the profession and its welfare. New procedure in this direction is now being developed. Your Board of Direction and its committees have been improving their work in this field and approaching the limits of what can be done by central bodies. To be fully effective, further development must be carried out by the entire membership, wherever it may be situated. Local Sections will find need to form committees on national affairs and to work very closely with the new National Affairs Committee of the Society. This latter is developing action procedure intended to give force to, and to seek results under, the broader policies which the Society is, from time to time, approving. It is not at all unlikely that some of the more forward-looking of our younger men may be called upon to join panels which will present the engineers' viewpoint to legislators and administrators. This would seem to be particularly indicated where the problem is one of improving the economic status of the younger man.

I trust that in this account of the older engineer's stewardship of the welfare of our profession you find cause for agreement that, while we have given less than the desired attention to that subject, this field of endeavor has not been wholly neglected by us, and that it is now given a high priority on the program of your Board. Further, we have developed a deep and sincere conviction of your ability to complete the job in a way that will be satisfactory to you, and to that end we are prepared to stay in the game even more effectively than heretofore. Full success requires your leadership.

It is my fond hope that as you ascend to that leadership, and as we turn it over to you, we both may profit by the experiences which we older men have had.

An excellent example of the advances that have been made toward obtaining for the engineering profession the recognition we all desire for it, and one which bids fair to open new avenues for recognized public service, is provided by the recent report on industrial disarmament of Japan which was sponsored by Engineers Joint Council. This council, as I am sure you know, is comprised of the presidents, junior past-presidents, and secretaries of the American Society of Civil Engineers, American Institute of Mining and Metallurgical Engineers, American Society of Mechanical Engineers, American Institute of Electrical Engineers, and American Institute of Chemical Engineers.

I am proud to recount that two ASCE members played important rôles in the preparation of this report and of the report for the disarmament of Germany, which preceded it, one as chairman of the National Engineers Committee, which prepared the report and the other as chairman of the Engineers Joint Council Executive Committee.

I am sure we are all sufficiently familiar with the nature of the report to recognize it as a major contribution of the engineering profession toward what we all hope will be lasting peace. Its transmittal to the State, War, and Navy departments attracted even more attention in the press than did its predecessor

report on the disarmament of Germany,[2] when it was made public by the National Engineers Committee of Engineers Joint Council last September.

What strikes me as particularly significant, however, and as a milestone in our progress toward the kind of public recognition that will extend the engineers' scope of usefulness to the public incalculably, is an editorial that appeared in *The New York Times* on June 15, within a day or two after the Japanese report was made public.

Under the heading, "A National Resource," *The Times* stated editorially:

> "The National Engineers Committee was appointed in February 1945, by representatives of the five national engineering societies at the request of the then Secretary of State, Mr. Stettinius, to bring the best American engineering thought to bear on the problem of the industrial disarmament of aggressor states. Last September it completed a report on Germany, the recommendations of which were substantially all included in the directive of the Allied Control Council of last March 29 for the limitation and control of German industry. Now its report on Japan has been made public in Washington, with the hearty endorsement of the State, War and Navy departments, as well as that of General MacArthur. It thus clearly foreshadows long-range American policy toward our defeated enemy in the Pacific.
>
> "The basic controls put forward for Japan are at variance with those proposed for Germany. That report based the broad primary control on the allocation of coal, since 85 per cent of Germany's electric power is generated by steam. In Japan, on the other hand, 73 per cent of her electrical energy at the wartime peak came from hydro-plants, which were both numerous and widely scattered. This necessitated a different approach to the problem. The engineers committee has recommended the regulation of the mineral and metals industries, the heavy chemical industries, the fuel industry and the construction industry, supplemented by the absolute prohibition of scientific research for military application, especially with regard to all forms of nuclear energy.
>
> "One of the most striking things about the present report is nowhere to be found in it, able document though it is. This is the availability for national purposes of such organized entities as the engineering profession, whose technical competence is matched by their freedom from any suspicion of political motivation. General Clay has acknowledged that the report on Germany has been his most effective instrument in dealing with both the British and the Russians. The latter, in particular, trust American technologists, whereas they look with suspicion upon our diplomats. Our own Government might do well to utilize even more fully such a national resource."

With such recognition, in its infant stage though it is, as a beginning, to what limitless heights may we not aspire for our profession as a whole? If the nontechnical segment of our population is commencing to visualize the engineering profession as "a national resource," must we not find inspirational hope for our profession in such stirrings of acceptance and recognition? As engineers, we have known for a long time, of course, that there are many additional services we can render. This latest and so clearly stated recognition of our potential by nontechnical men, coming as it does at a time when our younger men are so ready and able to assume their position in the affairs of our

[2] *Civil Engineering*, November, 1945, p. 524.

Society, seems to me to make for an ideal situation and to provide added stimulus to the ambitions of our younger men.

I am convinced that the time has come when the work in the field of professional improvement can no longer be that of a few committees, but must be the business of the entire membership, with much of the work done at the Local Section level, and by younger members. There you young men will find full opportunity to render service to the public, to the profession, and particularly to your generation of engineers.

Today you need not worry about a job. For the next several years there will be a scarcity of men to perform the great volume of engineering work waiting to be done. You will have a better opportunity than we had to select the line of work which most interests you. You will certainly find ready at hand obvious opportunity to assist in the improvement of the profession and of its social and economic status. If you put into that work the serious interest and the ability I see in you, the engineers of the succeeding generation can take pride in their place in society without reservation, and will call you blessed.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

STRENGTH OF THIN STEEL COMPRESSION FLANGES

Discussion

BY L. C. MAUGH AND L. M. LEGATSKI

L. C. MAUGH[25] AND L. M. LEGATSKI,[26] ASSOCIATE MEMBERS, ASCE.[26a]—For many years structural engineers have recognized the danger of subjecting thin elements to high compressive stresses; but, in general, they have avoided, rather than solved, the problems that are associated with the instability of such elements. However, industry is demanding that engineers remove the restrictions imposed by specifications and take advantage of any economy that may be obtained by the use of thin sheet metal. This interesting and timely paper is unusually important, therefore, inasmuch as it is related to the development of new specifications for the design of such structural members. Consequently, any discussion of the paper cannot logically ignore the problems that are presented, if design methods follow the procedures recommended by the author.

The basis of the solution recommended by Professor Winter is the equivalent width method, originally developed for the design of aircraft structures from tests on thin sheet panels with and without stiffeners. Most of these tests were made on specimens with large b/t-ratios and with E-values much less than those for steel. Consequently, the boundary effect was so large as compared to the primary resistance of the plate element that the latter effect could be ignored. The edge or boundary effect has commonly been evaluated (as the author states) by considering the strength of two-edge strips that are assumed to be unaffected by the central section between them. Even for such elements with large b/t-ratios, it would be more logical to consider each edge strip, in Fig. 1, as a separate plate rather than to combine the strips to form a single plate.

The principal question involved in the paper seems to be whether the equivalent width transformation is the best solution to the problem. Although the

NOTE.—This paper by George Winter was published in February, 1946, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1946, by Fred T. Llewellyn, and Jacob Karol; and October, 1946, by Robert L. Lewis and Dwight F. Gunder.

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[26] Research Engr., Dept. of Eng. Research, Univ. of Michigan, Ann Arbor, Mich.

[26a] Received September 19, 1946.

author rejects the Bryan formula[8] for calculating the resistance of the actual element, he does use it for determining the strength of the equivalent plate. Certainly, Eq. 2*b* is derived from the same basic differential equation as the general Bryan formula, that is,

$$\frac{\partial^4 \eta}{\partial x^4} + 2\frac{\partial^4 \eta}{\partial x^2 \partial y^2} + \frac{\partial^4 \eta}{\partial y^4} = -\frac{s}{D}\frac{\partial^2 \eta}{\partial y^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

in which η is the lateral displacement and $D = \dfrac{E t^3}{12(1-\mu^2)}$. Furthermore, there is no fundamental reason why the differential equation cannot be satisfied by superimposing the wave displacements involved in the edge resistance on

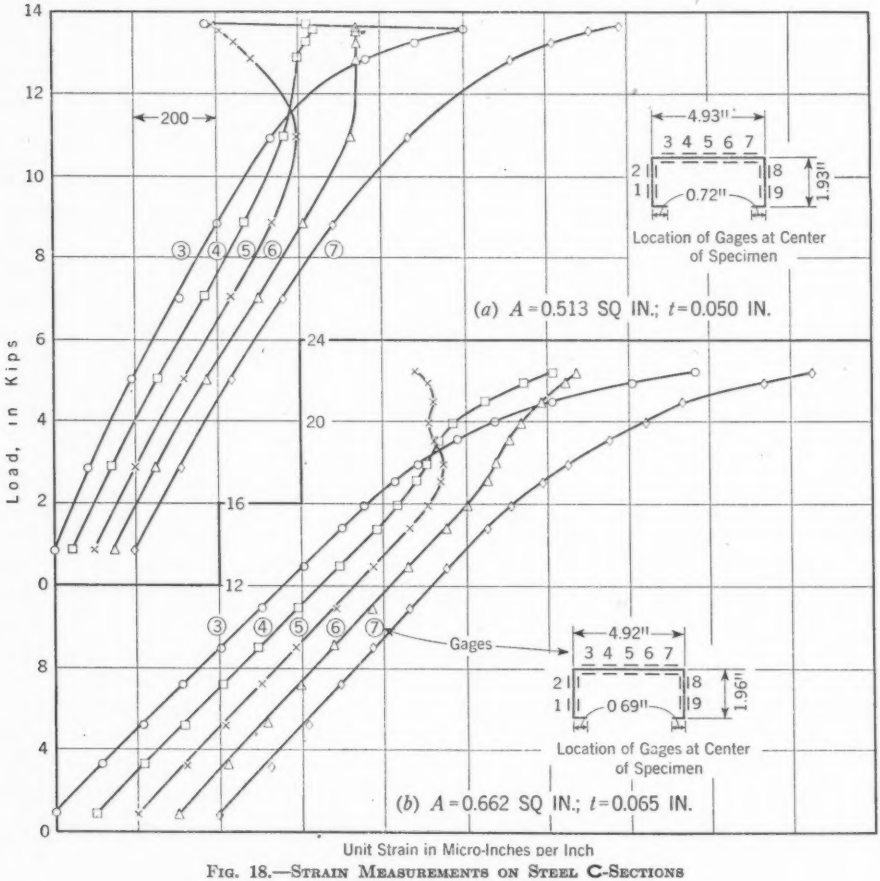

FIG. 18.—STRAIN MEASUREMENTS ON STEEL C-SECTIONS (L = 24 IN. AND L/r = 31.2)

the sine series ordinarily used for deriving the primary resistance of the plate.

The question arises, therefore, as to whether the ultimate strength of a thin element does not also depend on the primary resistance, as well as on the edge

[8] "On the Stability of a Plane Plate Under Thrust in Its Own Plane," by G. H. Bryan, *Proceedings*, London Mathematical Soc., Vol. 22, 1890, pp. 54–67

effect. Strain measurements have been made by various investigators which indicate a considerable increase in axial strain at the edges after the element has buckled. Strain measurements that were made by the writers on steel C-sections, 24 in. long, and that were tested in direct compression are shown in Fig. 18. These measurements were taken with standard electrical-resistance gages fastened in pairs to both faces of the plate elements and connected in series to give the average axial strain. In Fig. 18(*a*) where the web has a b/t-ratio of about 98, gage 5 at the center shows some reduction as the load approaches its ultimate value. Gages 4 and 6, adjacent to the center, remained practically constant, whereas gages 3 and 7 near the corners increased considerably. In Fig. 18(*b*) where the b/t-ratio of the web is about 75, gage 5 remains practically constant, gages 4 and 6 show some increase, and gages 3 and 7 show considerable increase as the ultimate strength of the specimen is approached. Because of the biaxial state of stress on elements near the center, the longitudinal stress is not directly proportional to the strain measurements shown. Nevertheless, the action in these specimens not only indicates an addition to the strain close to the corners but also shows that the primary resistance of the element is a definite and continuous phenomenon that may fluctuate but does not disappear as the ultimate strength of the specimen is reached. The author has not shown any distribution of strain across the compression flange of his specimens; nor has he stated whether such information was obtained.

A better approach would be to superimpose the edge effect on the primary resistance, whenever the element becomes unstable at a unit stress less than the proportional limit of the material. Above the proportional limit, a formula that considers the variation in E should be used and the edge effect neglected. For such assumptions, the ultimate strength of an element would be

$$s_u = s_c + s_e \qquad (21)$$

in which s_c is the critical stress of the element, and s_e is the average edge effect. It is assumed that s_c is less than s_p, the proportional limit. To determine s_e, the edge effect, it will be assumed that the resistance of the material near the boundary is approximated by that of a thin plate, supported at one edge and free at the other. The strength of this plate will be calculated from the usual formula; and, furthermore, the average unit stress on the edge plate will be taken equal to the proportional limit s_p. A mathematical investigation has indicated that this condition approximates closely the maximum resultant force that can be sustained by the assumed edge plate. Therefore, the width, b_s, of this edge effect will be such that:

$$s_p = \frac{K_1 E}{(b_s/t)^2} \qquad (22)$$

or

$$b_s = t\sqrt{\frac{K_1 E}{s_p}} \qquad (23)$$

The total additional resistance F' of both edges is, therefore,

$$F' = 2\,(s_p - s_c)\,t^2\,\sqrt{K_1}\sqrt{\frac{E}{s_p}} \quad \ldots\ldots (24)$$

and the average increase in stress over the element, due to the edge effect, is

$$s_e = \frac{F'}{b\,t} = \frac{2\,(s_p - s_c)\,\sqrt{K_1}}{\left(\frac{b}{t}\right)}\sqrt{\frac{E}{s_p}} \quad \ldots\ldots (25a)$$

The critical stress s_c is given by the Bryan formula:

$$s_c = \frac{K\,E}{\left(\frac{b}{t}\right)^2} \quad \ldots\ldots (25b)$$

The values of K and K_1 vary with the degree of restraint at the edges. For simply supported edges, $K = 3.62$ and $K_1 = 0.452$. These values give the following equation for the ultimate strength of an element that is simply supported at the edges:

$$s_c = \frac{3.62\,E}{\left(\frac{b}{t}\right)^2} \quad \ldots\ldots (26a)$$

and

$$s_u = s_c + \frac{1.35\,(s_p - s_c)}{\left(\frac{b}{t}\right)}\sqrt{\frac{E}{s_p}} \quad \ldots\ldots (26b)$$

It is suggested that the proportional limit, s_p, be taken as five eighths of the yield-point stress, s_y, to allow for variation of material in the specimens.

The values obtained from Eq. 26*b* have been compared with the experimental results that were obtained by L. Schuman and G. Back in their comprehensive series of tests. The theoretical values as shown by the solid lines[27] in Fig. 19 agree satisfactorily with the test results for large values of b/t, but are somewhat high for the smaller values. This condition is not surprising as the initial curvature in the specimens would have the greatest effect on the primary resistance of the stiffer elements. For steel plates with b/t-ratios less than 80, the edge effect could well be ignored. The dotted lines in Fig. 19 represent the critical stress s_c. The equations of the solid lines in Fig. 19 are, for $s_c > s_p$—

$$s_u = \frac{3.62\,E\,\frac{s_y}{s_y - s_p}}{\frac{3.62\,E}{s_y - s_p} + \left(\frac{b}{t}\right)^2} \quad \ldots\ldots (27)$$

[27] "Strength of Rectangular Flat Plates Under Edge Compression," by L. Schuman and G. Back, *Technical Report No. 356*, National Advisory Committee for Aeronautics, 1931, Figs. 9(*b*) and 10.

and, for $s_c < s_p$—

$$s_u = \frac{3.62\,E}{\left(\frac{b}{t}\right)^2} + \frac{1.35\,(s_p - s_c)\,\sqrt{E/s_p}}{\frac{b}{t}} \quad \text{.....(28)}$$

The author has based his design procedure on the use of Eq. 2*b* in which the coefficient C is represented by a function of $\sqrt{\frac{E}{s}} \times \frac{t}{b}$. To the writers, this

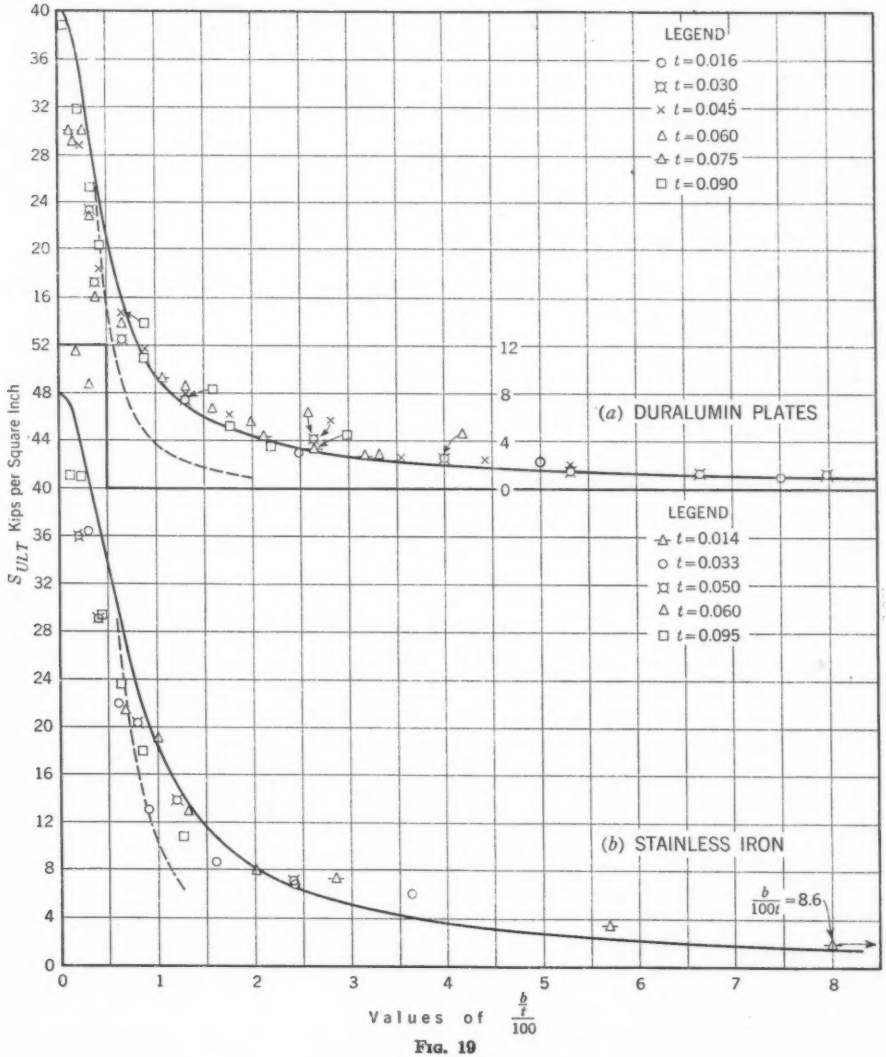

FIG. 19

variation is tantamount to assuming that the coefficient must correct the equation for other factors as well as for imperfections in the specimens and apparatus. The scatter of points in Fig. 2 could indicate many things besides

deviation from shape such as variation in edge restraint of the element, inaccuracies in the determination of the neutral axis and of stress distribution over the beam, and neglect of the primary resistance of the element.

Without accepting the effective width concept as the most logical approach to the stability problem in light-gage members, the writers would question several points in the author's method of determining the effective width experimentally. The instrumentation appears to be inadequate for the purpose of locating the neutral axis. With the gages on only one side of the member, as shown in Fig. 3, the effect of lateral bending and twisting action, which could introduce considerable error, is ignored. The fact that these effects were not nullified by duplicate gages on the opposite edge casts some doubt on the accuracy with which the neutral axis was located from strain measurements. The author states that at the failure load the neutral axis is again located from strain measurements. The only possibility of securing this result from the strains measured is by assuming a triangular strain distribution under conditions that approach the assumed rectangular stress distribution. For unsymmetrical sections such as those of test series A, the writers cannot agree that the neutral axis can be located accurately in this manner.

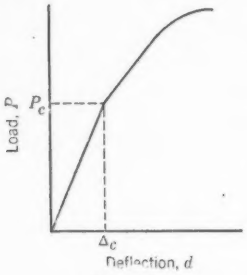

FIG. 20

In calculating deflections of light-gage beams, the use of an effective section appears to offer the best solution when the compression elements are stressed beyond the critical buckling stress. Fig. 20 shows a typical load-deflection curve for such a beam. The ordinate P_c indicates the load at which the flange elements reach the critical stress. Up to this load the properties of the full section should be used in calculating deflections. For higher loads an additional increment of deflection should be added to d_c, and this increment should be calculated using a reduced moment of inertia. For steel sections, sufficiently accurate results can be obtained by calculating the reduced moment of inertia using a flange width of $20\,t$ adjacent to each element that stiffens the flange.

Perhaps the author was led to the erroneous conclusion that the Bryan formula cannot be used to determine the ultimate buckling strength of thin plates—by attempting too close an analogy between the Bryan formula and the Euler formula for columns. Both formulas give a critical buckling load. The Euler load is the ultimate load for the column since there is no other element of strength to prevent failure. The Bryan load is the load at which the plate element forms into buckling waves, but it is not the ultimate load because a boundary or edge effect remains as an additional element of strength. If the Bryan load alone is compared to the ultimate load, the agreement is good for b/t-values up to about 50 for duralumin and 80 for steel, nickel, and monel metal. For larger b/t-values, the Bryan load is low (as indicated by the statement of Messrs. Schuman and Back, referred to by the author under the heading, "Type A. Compression Flanges Stiffened Along Both Longitudinal Edges: Equivalent Width of Thin Flanges"). This condition occurs because the boundary effect is small as compared to the effect of the Bryan load within the foregoing values of b/t, whereas, for larger b/t-values, the reverse is true.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

LANDSLIDE INVESTIGATION AND CORRECTION

Discussion

BY D. P. BARNES

D. P. BARNES,[17] M. ASCE.[17a]—A wide variety of landslides, together with an explanation of the success achieved by various remedial measures, is discussed in this interesting and practical paper.

The tendency of water to precipitate an earth movement can be explained in different ways. As in so many analytical matters, the choice of a suitable illustration with which to begin the analysis may mean the difference between a simple concept easily applicable to practical cases, or a complex concept with little useful application.

An illustration that appears to have some merit in providing a simple concept of the forces exerted by seepage water is that presented in Fig. 19. Con-

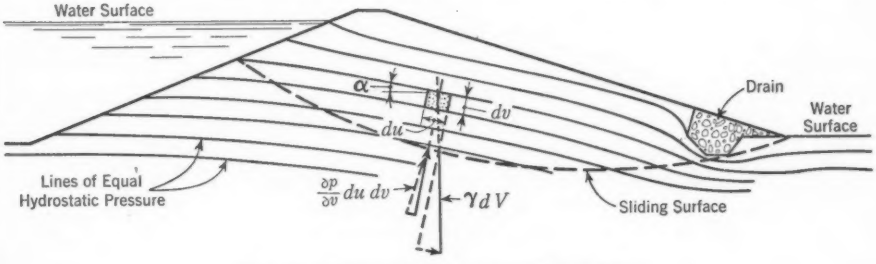

FIG. 19.—SEEPAGE FORCES IN AN EARTH DAM

sider the equilibrium of a soil prism of convenient length, du, and width, dv (thickness unity), with sides parallel and perpendicular, respectively, to lines of equal pressure. The soil in the prism is restrained on all sides by the contacts between soil particles and is acted upon by the hydrostatic pressures and its own weight. The water in the prism is acted upon by hydrostatic pressures,

NOTE.—This paper by Hyde Forbes was published in February, 1946, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1946, by T. W. Lambe; September, 1946, by Jacob Feld, George S. Harman, and Robert F. Legget; and November, 1946, by Rush T. Sill, Earl M. Buckingham, and Alfred V. Bowhay.

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[17a] Received October 21, 1946.

by the resistance of the soil particles to change in hydrostatic pressure, and by its own weight.

Isolating the prism, with its soil and water content intact (see Fig. 20), and applying the general equations of static equilibrium, the forces transmitted from the water to the soil are readily determined. For static equilibrium (that is, assuming that no masses are being accelerated), the sum of the components in any direction of all the forces acting on the body must be zero.

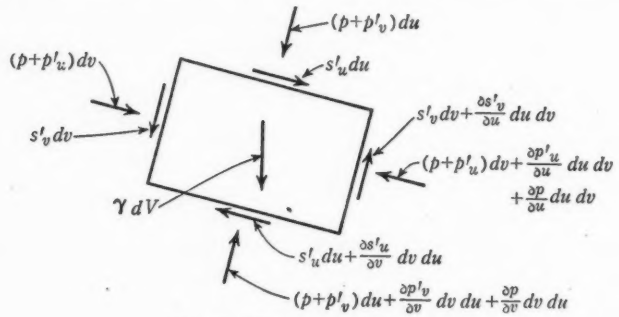

FIG. 20.—ELEMENTARY SOIL PRISM

Let p be the hydrostatic pressure; p' be contact pressure between soil particles; s' be the contact shearing stress between soil particles; V equal the volume of the soil prism; and γ equal to the wet unit weight of soil. Adding the components in the u-direction, therefore, yields:

$$\frac{\partial p'_u}{\partial u}\,du\,dv + \frac{\partial s'_u}{\partial v}\,dv\,du + \frac{\partial p}{\partial u}\,du\,dv + \gamma\,du\,dv\sin\alpha = 0 \quad \ldots\ldots (9a)$$

or, since $\frac{\partial p}{\partial u} = 0$,

$$\frac{\partial p'_u}{\partial u}\,dV + \frac{\partial s'_u}{\partial v}\,dV = -\gamma\sin\alpha\,dV \quad \ldots\ldots\ldots\ldots\ldots (9b)$$

Similarly, considering the components in the v-direction:

$$\frac{\partial p'_v}{\partial v}\,dv\,du + \frac{\partial s'_v}{\partial u}\,du\,dv + \frac{\partial p}{\partial v}\,dv\,du + \gamma\,dv\,du\cos\alpha = 0 \quad \ldots\ldots (10a)$$

or

$$\frac{\partial p'_v}{\partial v}\,dV + \frac{\partial s'_v}{\partial u}\,dV = -\frac{\partial p}{\partial v}\,dV - \gamma\cos\alpha\,dV \quad \ldots\ldots\ldots\ldots (10b)$$

The right-hand members of Eqs. 9*b* and 10*b* represent the forces acting on the prism due to gravity and to changes in the hydrostatic pressure. Adding the equations vectorially yields:

$$\left(\frac{\partial p'_u}{\partial u}\,dV + \frac{\partial s'_u}{\partial v}\,dV\right) \oplus \left(\frac{\partial p'_v}{\partial v}\,dV + \frac{\partial s'_v}{\partial u}\,dV\right)$$
$$= -\gamma\sin\alpha\,dV \oplus \left(-\frac{\partial p}{\partial v}\,dV - \gamma\cos\alpha\,dV\right) \quad \ldots\ldots\ldots\ldots (11)$$

The right-hand and left-hand members, however, need be concurrent only if $\frac{\partial s'_u}{\partial v} = 0$, and $\frac{\partial s'_v}{\partial u} = 0$, which is unlikely. (The sign $\oplus$ in Eq. 11 indicates a vector sum.)

In making a graphical analysis, it is convenient nevertheless to neglect $\frac{\partial s'_u}{\partial v}$ and $\frac{\partial s'_v}{\partial u}$. The error of this procedure is of the same character as that introduced by assuming, in the absence of percolating water, that the weight of the soil is transmitted vertically downward indefinitely in the same line of action. As Karl Terzaghi,[18] M. ASCE, has shown, its effect is only to cause a slight redistribution of pressures without affecting their total. For practical purposes, therefore, the increment in soil pressure or contact pressures through the soil prism, $du\ dv$, is equal to, and has the opposite direction to, the vector sum comprising the right-hand member of Eq. 11, and may be considered to act through the center of gravity of the prism. The simplicity of this approach arises from the fact that in any rectangular element it is necessary to deal with only the wet weight, $\gamma\ dV$, acting downward and the hydrostatic pressure difference, $\frac{\partial p}{\partial v}\ du\ dv$, acting normal to the lines of equal pressure.

To obtain the total pressures between soil particles along a typical sliding surface in an embankment, therefore, it is necessary only: (*a*) To divide the area intercepted by the sliding arc into convenient finite rectangular elements, (*b*) to determine the forces exerted by the hydrostatic pressure difference $\left(\text{corresponding to } \frac{\partial p}{\partial v}\ dV\right)$ from the equal pressure lines and the forces of gravity from the wet weights, and (*c*) to add their several effects vectorially. In Fig. 19, the forces $\frac{\partial p}{\partial v}\ du\ dv = \frac{\partial p}{\partial v}\ dV$ and $\gamma\ dV$ are shown projected to the sliding surface and there resolved into their tangential and normal components.

It would seem that the mere presence of water, except in so far as it may affect the cohesion or coefficient of friction within the soil mass, is not a direct indication of an increased susceptibility to sliding. For example, on the upstream slope of an earth dam through which water is percolating toward the downstream drain, the action of the water is to shift the resultant of all the forces acting on a prism of soil slightly downstream from vertical, and, thus, to reduce the components tending to produce sliding in the upstream face. The downstream face, however, toward which the water is flowing, has an increased tendency to slide as a result of this same shift in the resultant of the forces acting on each soil prism.

The case of submergence without flow can be deduced by an inspection of Fig. 19 if it is imagined that the downstream water surface is raised to the level of the upstream surface, and that the lines of equal hydrostatic pressure are correspondingly rotated to their normal horizontal position as found in any body of static water. The resultant forces on the soil prism become con-

[18] "The Mechanics of Shear Failures on Clay Slopes and the Creep of Retaining Walls," by Karl Terzaghi, *Public Roads*, December, 1929.

current and vertical, and the forces tending to oppose sliding, while reduced, bear the same relation to the frictional resistance to sliding as before.

Most cases with which the engineer or geologist is likely to deal, with the sole exception of artificial embankments constructed for the purpose of impounding water, will, nevertheless, probably resemble the examples cited by Mr. Forbes in that the presence of water will contribute significantly toward the danger of movement. It may well be, also, that, although an approximate analysis of the case of static submergence indicates no increase in susceptibility to sliding, for many practical construction problems such as those encountered in the installation of piers or underwater footings, the secondary factors such as reduction in cohesion or in coefficient of friction and swelling may outweigh abstract conclusions. Any predictions with respect to susceptibility of a completely inundated embankment to sliding should, therefore, be attempted with caution.

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DISCUSSIONS

THRUST EXERTED BY EXPANDING ICE SHEET

Discussion

BY ERNEST BROWN, B. R. MCGRATH, AND ROBERT E. KENNEDY

ERNEST BROWN,[3] ESQ.[3a]—A welcome addition to the knowledge of factors that determine the pressure exerted by an expanding sheet of ice is presented in this paper. The writer first became interested in the fundamental question of the strength of ice in 1927 when he conducted tests for the Joint Board of Engineers on the St. Lawrence Waterway, and prepared the Appendix to the report, dealing with the strength of ice under different conditions. Some few years later, when plans were being made for the building of a hydroelectric plant on the Churchill River, the question of selecting a well-founded value for the ice thrust against the dam became urgent. Values as high as 45,000 lb per lin ft were recorded as having been used. Such a large value would indicate extremely large ice-pressure moments as compared with water-pressure moments, since the head was about 50 ft. The plant, if constructed, would be at the most northerly site yet developed in Canada. Absence of roads made it necessary to bring all materials to the site over frozen lakes and rivers during the winter months, and the economic aspect of the project depended largely on the question of the value of the ice thrust under probable conditions at the site. There seemed to be good reason to doubt the existence of such extremely high pressures as had been used.

However, the data from the waterways tests could not be used to furnish a dependable value for thrust likely to develop from a prolonged and gradual rise of temperature, although they supported the opinion that such high pressures as that mentioned above would probably not arise. Therefore, the writer, in association with G. C. Clarke, M. ASCE, conducted laboratory tests under controlled conditions in an effort to simulate the conditions in nature. The results were reported in a joint paper to the Engineering Institute of Canada in 1932, and along with the waterways tests, are listed in the Bibliography of the paper (3)(6).[3b] Mr. Rose has used these results in preparing the curves shown in Fig. 2, which give estimates of the thrust for ice sheets of different

NOTE.—This paper by Edwin Rose was published in May, 1946, *Proceedings*.

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[3a] Received September 13, 1946.

[3b] Numerals in parentheses, thus: (3), refer to corresponding items in the Bibliography (see Appendix of the paper), and at the end of discussion in this issue.

thicknesses, considering different rates of air temperature rise and restraint. He has applied the principles of heat transmission to estimate the temperature gradients within the ice sheet under prescribed conditions, and has used the data found by the writer and Mr. Clarke in the Churchill River tests in conjunction with the computed gradients to give the total thrust, using a process of summation.

Each individual project presents its own special conditions. The designer must estimate the probable thickness of ice sheet, nature of snow cover, rate of air temperature rise, restraint at the site, etc., and then must decide on a value for the thrust to be used. He must consider the question of the depth to which temperature changes of the atmosphere may penetrate the ice sheet, since the temperature gradient is an important factor. He may not be able to satisfy his desire for precise data, and will have to use his best judgment after a careful study of all available information. This procedure was followed for the Churchill River project, and no attempts were made to compute temperature gradients in the ice sheet.

It would be helpful if the author would give, in some detail, the calculations for a typical temperature gradient, and submit some selected curves showing the estimated gradients on which the thrusts in Fig. 2 are based. The applicability of the curves might be judged better if the engineer were enabled to consider the degree of agreement between the computed gradients for a few typical conditions, and the gradients that appear to be reasonably likely to occur when the available data for a particular project have been thoroughly studied. It is interesting to note that the curves in Fig. 2, although covering a very wide set of assumed conditions, do not indicate the likelihood of the existence of such high pressures as have been used in some designs. The writer believes that pressures as great as 50,000 lb per lin ft are imaginary and fantastic. Mr. Rose's data seem to be better founded.

The writer's experience in the testing of ice in a laboratory suggests that experiments on temperature gradients would be very difficult, whether under controlled conditions or in a natural sheet of ice. However, at some future date experiments will be made and will form another link in the chain of knowledge. In the meantime, some typical examples of the gradients used in preparing Fig. 2 would add greatly to the valuable study of the problem which Mr. Rose has presented so ably.

The writer regrets that it has not fallen to his lot to pursue the work on the Churchill River project further. It is gratifying to find that the results then obtained have been used so effectively in the broad general study made by Mr. Rose.

B. R. McGRATH,[4] ESQ.[4a]—The determination of the thrust exerted by an expanding ice sheet is a complex problem because of the plastic behavior of the ice when subjected to temperature increases, and because of various conditions of restraint. There are also a number of influencing factors in nature which affect the rate of temperature change and the degree of restraint encountered

[4] Engr., Bureau of Reclamation, Denver, Colo.

[4a] Received September 30, 1946.

at a specific location. The paper by Mr. Rose offers an excellent approach to the solution of this problem. Estimated ice pressures obtained by this analysis are believed to be sufficiently conservative for good practice in the design of hydraulic structures.

As suggested by the author's statement (see paragraph following Eq. 2), on June 22, 1942, the writer, with the aid of valuable suggestions from Douglas McHenry (20), submitted a memorandum to the chief designing engineer of the Bureau of Reclamation setting forth a method for computing ice pressures based on:

(*a*) The experimental data of Messrs. Brown and Clarke on ice pressures due to temperature increases (3);

(*b*) An adaptation to ice sheets of the graphical method developed by E. Schmidt for obtaining temperature gradients within a mass for one-dimensional heat flow (11); and

(*c*) An evaluation of the effects of lateral restraint according to the theory of elasticity.

The preliminary nature of the work was emphasized in the memorandum, and it was recommended that additional studies be made to consider several factors entering into the problem. Shortly thereafter, the writer was called to active duty in the Civil Engineering Corps of the U. S. Navy and was unable to continue work on the problem.

The purpose of the study was to devise some simple usable curves for evaluating ice pressures against hydraulic structures. It was prepared from a further interpretation of results obtained by Messrs. Brown and Clarke as cited by Mr. Rose (3). These writers (3*b*) called attention to two important factors in the problem of determining ice pressure on a dam—the rate and extent to which a rise in air temperature will be transmitted through the ice cover, and the effect of restraining the ice in a direction parallel to the dam.

An approach to the solution of the first of these problems began with the first three of the assumptions mentioned by Mr. Rose in Section 4. Using a graphical method developed by E. Schmidt (11) and described by Douglas McHenry (20), and data presented by Messrs. Brown and Clarke (3), the writer prepared the pressure diagrams shown in Fig. 4. They were taken from several temperature gradients, at known time intervals, and within a normal range of ice thicknesses. The rate of temperature rise was determined at different depths within the ice mass for time intervals computed from Eq. 1, with a diffusivity constant h^2 for ice equal to $0.0481 \frac{\text{ft}^2}{\text{hr}}$. Designating θ_p, θ_q, and θ_r as the temperatures at three successive space intervals on a gradient curve at a given time, the writer developed substantially the formula cited by Mr. Rose as Eq. 2.

With the help of Fig. 4 equivalent ice pressures σ were found at each space interval for each temperature time interval. The total pressure acting within the ice was summed at each space interval for the total time intervals involved, and pressure curves were plotted showing the distribution of total pressure within the ice sheet.

The effect of the lateral restraint of the ice in a direction parallel to the dam may be estimated by first assuming a differential element of ice such as that shown in Fig. 5.

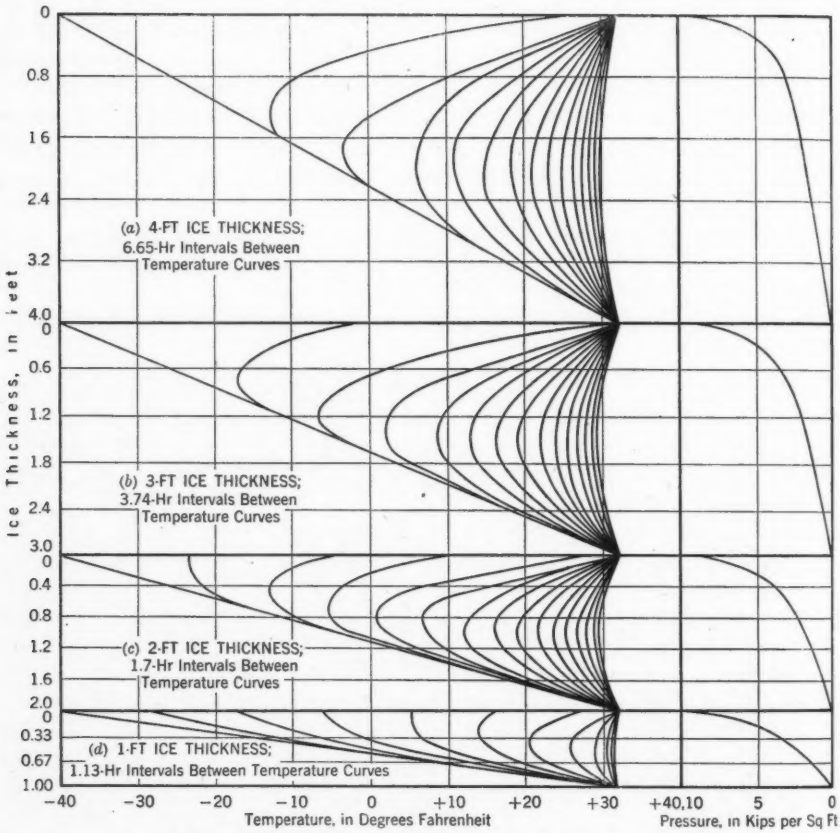
Ice Thickness, in Feet
(a) 4-FT ICE THICKNESS; 6.65-Hr Intervals Between Temperature Curves
(b) 3-FT ICE THICKNESS; 3.74-Hr Intervals Between Temperature Curves
(c) 2-FT ICE THICKNESS; 1.7-Hr Intervals Between Temperature Curves
(d) 1-FT ICE THICKNESS; 1.13-Hr Intervals Between Temperature Curves
Temperature, in Degrees Fahrenheit
Pressure, in Kips per Sq Ft

FIG. 4.—ICE TEMPERATURE CURVES AND RESULTING PRESSURE CURVES

For static equilibrium:

$$\xi_x = \frac{1}{E}[\sigma_x - \mu(\sigma_y - \sigma_z)] + \alpha\,\Delta\theta \quad \ldots\ldots (3a)$$

$$\xi_y = \frac{1}{E}[\sigma_y - \mu(\sigma_x - \sigma_z)] + \alpha\,\Delta\theta \quad \ldots\ldots (3b)$$

and

$$\xi_z = \frac{1}{E}[\sigma_z - \mu(\sigma_x - \sigma_y)] + \alpha\,\Delta\theta \quad \ldots\ldots (3c)$$

in which ξ_x, ξ_y, and ξ_z are unit strains and σ_x, σ_y, and σ_z are unit stresses in the x, y, and z directions, respectively. Furthermore, α is the coefficient of expansion; $\Delta\theta$ is the temperature change; E is Young's modulus; and μ is Poisson's ratio.

For complete restraint in the x-direction and the y-direction and no restraint in the z-direction:

$$\xi_x = \xi_y = 0; \qquad \sigma_x = \sigma_y; \qquad \text{and } \sigma_z = 0 \ldots\ldots\ldots\ldots (4)$$

From Eqs. 3a or 3b:

$$\sigma_y - \mu\,\sigma_y + E\,\alpha\,\Delta\theta = 0 \ldots\ldots\ldots\ldots\ldots\ldots (5a)$$

or,

$$\sigma_y = -\frac{E\,\alpha\,\Delta\theta}{1-\mu} \ldots\ldots\ldots\ldots\ldots\ldots (5b)$$

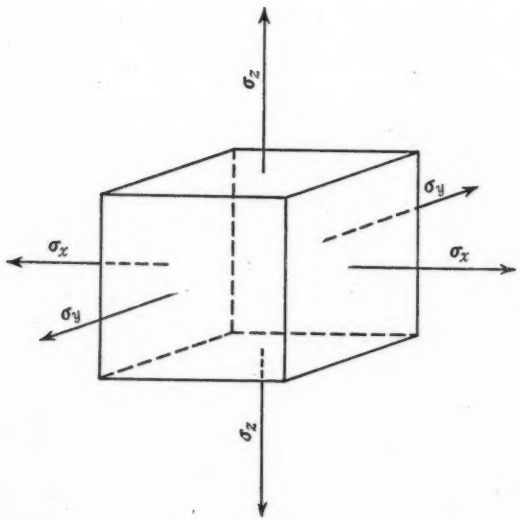

FIG. 5

Let σ'_y represent the stress due to restraint in the y-direction only; then

$$\sigma'_y = -E\,\alpha\,\Delta\theta \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

and the difference ($\Delta\sigma_y$) between σ_y and σ'_y is

$$\Delta\sigma_y = -E\,\alpha\,\Delta\theta\,\frac{1}{1-\mu} + E\,\alpha\,\Delta\theta \ldots\ldots\ldots\ldots\ldots\ldots (7a)$$

or,

$$\Delta\sigma_y = -E\,\alpha\,\Delta\theta\,\frac{\mu}{1-\mu} \ldots\ldots\ldots\ldots\ldots\ldots (7b)$$

The ratio of increase of stress for complete lateral restraint then becomes

$$\frac{\Delta\sigma_y}{\sigma'_y} = \frac{-E\,\alpha\,\Delta\theta\,\frac{\mu}{1-\mu}}{-E\,\alpha\,\Delta\theta} = \frac{\mu}{1-\mu} \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

If $\mu = 0.25$, then $\frac{\Delta\sigma_y}{\sigma'_y} = \frac{1}{3}$; and if $\mu = 0.5$, $\frac{\Delta\sigma_y}{\sigma'_y} = 1$. It is thought that the

two assumed values of $\mu = 0.25$ and $\mu = 0.5$ may be taken as limits, and that the stress for a condition of complete lateral restraint should be increased over that of no lateral restraint by a ratio somewhere between the values of $\frac{1}{3}$ and 1.

Fig. 6 shows ice pressures for thicknesses that would be likely to occur in the reservoir. Areas of pressure curves in Fig. 4 were plotted as points when constructing the curve for the condition of no lateral restraint. These points were then increased by $\frac{1}{3}$ and by 1, and curves were drawn corresponding to the limits mentioned. Table 1 is a comparison of ice-pressure values obtained from Fig. 6 and those obtained from curve B, Fig. 2(*a*). The values check within $3\frac{1}{2}\%$ for a condition of no lateral restraint. For complete lateral restraint, the maximum difference between the average of the two limiting values of Poisson's ratio used by the writer and curve B, Fig. 2(*a*), is approximately 2%.

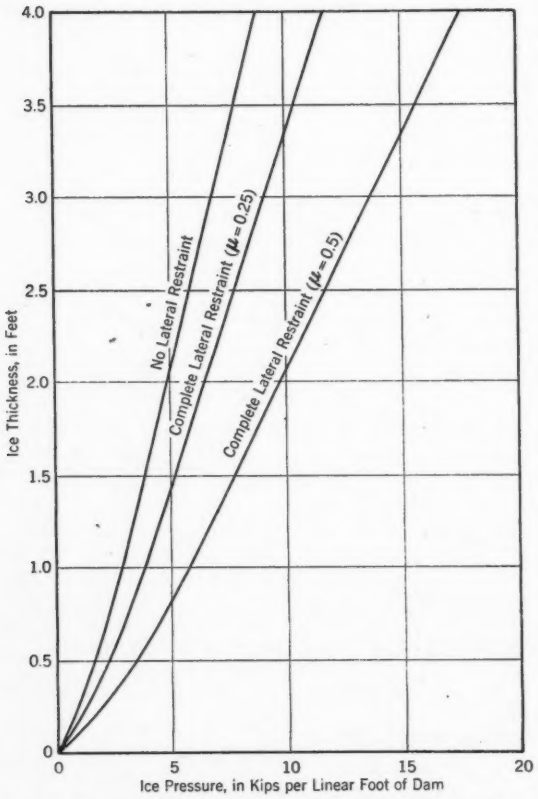

FIG. 6.—ICE PRESSURE CURVES FOR CONDITIONS OF NO LATERAL RESTRAINT AND COMPLETE LATERAL RESTRAINT, PLOTTED AGAINST ICE THICKNESS

Except in unusual cases, it is believed that ice pressures based on a temperature change of 5° F per hr would be satisfactory for design purposes especially for ice thicknesses greater than 2 ft. This is illustrated by a study of Fig. 4. After rising from −40° F to +32° F, the air temperature then remains constant during a considerable length of time while the temperature of the ice rises to this level. For an ice thickness of 2 ft this period is approximately 17 hours. Likewise, an ice thickness of 4 ft requires approximately 80 hours of constant air temperature at +32° F to permit the total pressure to develop.

It would seem very unlikely that such an assumption of constant temperature at this particular level would actually take place in nature immediately following a relatively short period of constant rise from −40° F. Some fluctuation of air temperature, either upward above freezing or downward, during

this period would seem most likely with a consequent reduction in ice pressures over those obtained in the analysis.

Further study of the problem would probably reveal that fluctuations of temperature tend to affect the pressures developed within an ice sheet in a manner somewhat similar to a relatively slower rate of constant temperature changes.

TABLE 1.—COMPARISON OF ICE PRESSURES, IN THOUSANDS OF POUNDS (KIPS) PER LINEAR FOOT

Ice thickness (ft)	Fig. 6				Curve B, Fig. 2(a)	
	No lateral restraint	Complete Lateral Restraint			No lateral restraint	Complete restraint
		$\mu = 0.25$	$\mu = 0.5$	Average $\mu = 0.375$		$\mu = 0.365$
4	8.8	11.7	17.7	14.7	9.1	14.4
3	6.9	9.1	13.7	11.4	7.1	11.2
2	4.9	6.5	9.7	8.1	5.0	7.9
1	2.9	3.9	5.8	4.85	2.9	4.4

Another important factor entering into the solution of the problem is the variation to be expected in Poisson's ratio for ice due to changes in its plasticity at different temperatures, the higher Poisson's ratio values occurring at higher temperatures. A search made of available publications on the subject of ice pressures reveals no practical data for the determination of this effect.

The writer desires to suggest, without detracting in any way from the valuable presentation by Mr. Rose, that much remains to be accomplished which might supplement the work already done. Valuable contributions could be made toward a more exact analysis and a better understanding of the ice pressures likely to occur at given reservoir locations through:

(1) Field measurements of ice pressures with their accompanying temperature records taken at various reservoirs, whereby the degree of restraint exerted on the ice sheet could be predicted for a design problem;

(2) Analyses based on climatological studies of various locations within the northern states; and

(3) Laboratory tests for the determination of Poisson's ratio variations with ice-temperature changes.

ROBERT E. KENNEDY,[5] M. ASCE.[5a]—In Section 10 of his paper the author mentions unpublished studies on ice thickness on reservoirs of Bureau of Reclamation projects. These studies were initiated in 1939 through the Office of the Chief Engineer. The readings were taken once a week in most cases and continued for three or four years. Since this is a limited period in which to determine any kind of maximum, another request was sent to the field offices in December, 1944, asking for any other reliable records taken from memory or otherwise covering longer periods. All results are incorporated in Table 2.

[5] Bureau of Reclamation, Boulder City, Nev.

[5a] Received October 14, 1946.

TABLE 2.—Ice Thickness on Reservoirs and Lakes, Bureau of Reclamation

Reservoir and dam	Project	Latitude (north)	Water surface[a] (ft)	Winters of record	Ice-cover period[b]	Maximum Thickness		
						In.	Month	Year
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
California:								
Boca	Truckee storage	39° 20′	5,590	1939–1942	Dec.–Apr.	17	Feb.–Mar.	1942
Clear Lake	Klamath	41° 55′	4,530	1939–1942	Dec.–Mar.	10	Jan.	1942
Colorado:								
Grand Lake[c]	Colorado-Big Thompson	40° 15′	8,365	1920–1944	Dec.–May	25½	Mar.	1942
Mary's Lake[c]		40° 23′	8,022	1941–1942	Nov.–Apr.	18	Mar.	1942
Taylor Park	Uncompahgre	38° 50′	9,292	1939–1943	Nov.–May	26	Jan.–Apr.	1943
Idaho:								
American Falls	Minidoka	42° 50′	4,345	1939–1943	Dec.–Apr.	16	Mar.	1942
Arrowrock	Boise	43° 30′	3,150	1925–1945	Dec.–Apr.	25	....	....
Black Canyon	Boise	44° 0′	2,480	1931–1944	Oct.–Mar.	14	....	....
Deadwood	Boise	44° 15′	5,315	1939–1944	Dec.–Apr.	26	Mar.	1942
Island Park	Upper Snake River	44° 24′	6,295	1939–1942	Nov.–May	14	Jan.	1941
Lake Wolcott[d]	Minidoka	42° 40′	4,240	1939–1943	Dec.–Mar.	14	Feb.	1942
Montana:								
Bowdoin Lake[e]	Milk River	48° 25′	2,205	1939–1942	Oct.–Apr.	26	Feb. 1940	Mar. 1941
Fresno	Milk River	48° 35′	2,540	1939–1942	Nov.–Apr.	23	Mar.	1940
Gibson	Sun River	47° 35′	4,665	1939–1941	Nov.–Apr.	16	Feb.–Mar.	1940
Lower St. Mary's Lake	Milk River	48° 45′	4,600	1939–1942	Nov.–Apr.	21½	Mar.	1940
Nelson	Milk River	48° 30′	2,210	1939–1942	Nov.–Apr.	29	Feb.	1941
Sherburne Lakes	Milk River	48° 50′	4,750	1939–1942	Nov.–Apr.	22½	Mar.	1942
Nevada:								
Rye Patch	Humboldt	40° 30′	4,100	1936–1944	Dec.–Jan.	12	....	....
Oregon:								
Agency Valley	Vale	44° 0′	3,330	1939–1942, 1945–1946	Dec.–Mar.	13	Feb. 1941	Feb. 1946
Gerber	Klamath	42° 10′	4,820	1939–1942	Dec.–Apr.	15	Mar.	1942
Owyhee[e]	Owyhee	43° 30′	2,600	1932–1944	Feb.	0	....	....
Thief Valley	Baker	45° 0′	3,100	1939–1942	Nov.–Mar.	14	Jan.	1942
Unity	Burnt River	44° 30′	3,800	1939–1942	Dec.–Apr.	14½	Feb.	1942
Upper Klamath Lake[f]	Klamath	42° 15′	4,140	1939–1942	Dec.–Mar.	10½	Jan.	1942
South Dakota:								
Belle Fourche	Belle Fourche	44° 40′	2,950	1924–1944	Nov.–Apr.	31	Jan.	1928
Utah:								
Deer Creek	Provo River	40° 30′	5,318	1941–1945	Dec.–Apr.	18	....	....
Hyrum	Hyrum	41° 40′	4,655	1939–1942, 1945	Dec.–Apr.	20	Mar.	1942
Moon Lake	Moon Lake	40° 33′	8,115	1939–1943	Nov.–May	25	Mar.	1943
Pine View	Ogden River	41° 15′	4,845	1939–1942	Nov.–Apr.	8	Feb.	1942
Washington:								
Bumping Lake	Yakima	46° 50′	3,410	1939–1943	Dec.–Apr.	15	Feb.	1943
Lake CleElum[g]	Yakima	47° 15′	2,185	1929–1944	Jan.–Apr.	10	Feb.	1929
Lake Kachess	Yakima	47° 15′	2,232	1939–1943	Nov.–Apr.	12	Mar.	1942
Lake Keechelus	Yakima	47° 18′	2,480	1939–1943	Dec.–Apr.	14	Jan.–Apr.	1942
Tieton	Yakima	46° 40′	2,880	1939–1943	Dec.–Apr.	13	Mar.	1942
Wyoming:								
Alcova	Kendrick	42° 30′	5,420	1939–1942	Nov.–Apr.	16½	Feb.	1940
Bull Lake	Riverton	43° 15′	5,765	1919–1944	Dec.–Apr.	26	....	1928–1929
Diversion Dam[h]	Riverton	43° 15′	5,569	1919–1944	Nov.–Mar.	36	....	1928–1929
Grassy Lake	Upper Snake River	44° 8′	7,150	1939–1941	Oct.–May	16	Jan.	1941
Guernsey	North Platte	42° 15′	4,410	1939–1940, 1941–1942	Dec.–Mar.	17	Feb.	1940
Jackson Lake	Minidoka	43° 50′	6,750	1939–1942	Dec.–May	30	Mar.	1941
Ocean Lake	Riverton	43° 15′	5,216	1928–1944	Nov.–Apr.	28	....	1928–1929
Pathfinder	North Platte	42° 30′	5,780	1940–1942	Nov.–Apr.	23½	Feb.	1941
Pilot Butte	Riverton	43° 15′	5,440	1925–1944	Nov.–Apr.	28 to 30	....	1928–1929
Seminoe	Kendrick	42° 10′	6,250	1939–1942	Nov.–Apr.	24½	Mar.	1942
Shoshone	Shoshone	44° 30′	5,280	1933–1944	Dec.–Apr.	22	....	1934 and 1937

[a] Average water-surface elevation, in feet. [b] Maximum range. [c] No dam. [d] Minidoka Dam. [e] No ice in the winter of 1940–1941. [f] Link River Dam. [g] Approximately 0.5 in. of ice in February, 1942. [h] Diversion Dam on Wind River.

A multiple correlation was made of observed maximum ice thickness with latitude and elevation. The correlation R of 0.479 with a standard error of 0.120 indicates a highly significant relationship. Significance is accorded a correlation of thirty or more items when R is at least twice its standard error, often called σ. High significance exists when the correlation is at least three times the value of σ.

The formula resulting from the correlation is

$$L_{max} = 1.15\,\phi + 2.58\,Z - 43.4 \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in which L_{max} is estimated maximum thickness in inches, ϕ is latitude in degrees, and Z is elevation in thousands of feet. The error of the estimate is 6.7 in., meaning that if Eq. 9 were used to compute the thickness for comparison with the observed value, about one third of the time the computed thickness would be larger or smaller than the observed thickness, by about 7 in. or more. Actually, it proved to be outside the ±7-in. range in thirteen of the forty-five readings.

The author uses ice thickness in units of a foot, and areas of pressure in terms of square feet. Eq. 9 could be expected to predict thicknesses that would fall within about a half foot of the observed value about two thirds of the time. That prediction is limited to places within latitudes 39° N. to 49° N., and elevations from 2,000 ft to 9,000 ft in western United States west of the 105th meridian, and for such meteorological conditions as existed mainly between 1939 and 1943. Extrapolation beyond the limits of the original data is always uncertain. The author cites a thickness of 5 ft observed in northern Canada. To compute such a thickness one must go as far north as seems probable, say, latitude 65° N., and to an unlikely elevation of 10,000 ft.

Eq. 9 has additional limitations. Although the correlation coefficient of 0.48 is highly significant with respect to its standard error, it, itself, is not high. In fact, it is so low that the corresponding coefficient of alienation is uncomfortably high. It raises doubts as to the efficacy of the formula. The alienation coefficient is $(1 - R^2)^{0.5}$ and amounts to 0.88. It measures the effect of omitting pertinent factors that contribute to the formation of ice. It would appear that more effective factors are omitted than are included.

To illustrate some of the omissions, reference is made to the larger items of heat in the energy equation. Heat contributed to a water surface comes mainly from two sources, the infrared wave lengths of solar energy and the latent heat released when ice is formed. Heat lost is mainly the outgoing black body radiation which figures so evidently in the freezing of ice on clear cold nights.

Latitude and elevation enter directly into the energy equation only in connection with solar radiation. Even there cloudiness is an important factor and it is omitted from Eq. 9.

Latitude and elevation have only an indirect effect on outgoing black body radiation which is primarily a function of surface air temperature expressed in degrees absolute. Eq. 9 does not include air temperature.

Omitted from Eq. 9 is the effect of atmospheric moisture in gaseous state but not in clouds. This moisture radiates back to earth in amounts 30% to

60% of that of a black body. Again the presence of low or middle clouds reduces outgoing radiation also.

Other factors not related at all closely to latitude and elevation are loss of heat by sublimation, conduction, convection and wind movement, and the saving of heat by snow cover.

Even with these limitations, Eq. 9 may be adequate for the problem in hand. The formation and dissipation of ice by the energy equation is another subject.

Bibliography.—

(3) **"Ice Thrust in Connection with Hydro-Electric Plant Design,"** by E. Brown and G. C. Clarke, *The Engineering Journal*, January, 1932, pp. 18–25. (*a*) Fig. 8, p. 24. (*b*) p. 25.

(6) **"Report of Joint Board of Engineers on the St. Lawrence Waterway,"** St. Lawrence Waterways Projects, Ottawa, Ont., Canada, 1927, Appendix E, Ice Formation on the St. Lawrence, and Appendix F, Experiments on the Strength of Ice.

(11) **"A Föppl's Festschrift,"** by E. Schmidt, J. Springer, Berlin, 1924, p. 179.

(20) **"The Control of Temperature in Massive Concrete Structures,"** by Douglas McHenry, Am. Inst. of Physics, 1941, Volume entitled, "Temperature; Its Measurement and Control in Science and Industry," Reinhold Publishing Corp., p. 912.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

MODEL STUDY OF BROWN CANYON DEBRIS BARRIER

Discussion

BY ETTORE SCIMEMI, AND J. W. JOHNSON

ETTORE SCIMEMI,[6] ESQ.[6a]—Among the problems treated in this most interesting paper is the depth of the scour at the foot of a spillway dam such as Brown Canyon.

Experiments were performed on a model and since, in the prototype, the material encountered was a fairly erosive bedrock, the authors were obliged to substitute for this material, in the model, some other material with similitude characteristics reduced in the scale 1:50. Such a transformation presents considerable difficulties, not only in the selection of a suitable material, but in obtaining or developing one that will have the proper size and size distribution. In a reduction of 1:50 it is inevitable that the effect of some of the finer material in the prototype will be lost entirely in the model. In addition to these difficulties (which could make the results questionable), there are some general considerations in the design of a spillway dam.

A nappe that falls vertically from the top of a dam, and which can scour the bottom of the river, cuts the channel to a depth that depends on the balance of the forces engaged; that is, equilibrium between energy of the nappe and the energy destroyed in the whirlpool. Therefore, the volume and the depth of the scour are not more rigorous functions of the size of the material on the bottom, but only a function of the difference in the elevation of the overflowing nappe and that of the stilling pool below. Alessandro Veronese[7] examined this problem, and gave the following formula for the depth (in metric units) of the scour y_s:

$$y_s = 1.90\, H^{0.225} q^{0.54} \ldots\ldots\ldots\ldots\ldots\ldots (2a)$$

NOTE.—This paper by Karl J. Bermel and Robert L. Sanks was published in May, 1946, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1946, by Clifford A., Betts, and Roger E. Amidon.

[6] Prof., Hydr. Inst., Univ. of Padua, Padua, Italy.

[6a] Received August 16, 1946.

[7] "Erosioni di fondo a valle di uno scarico," by A. Veronese, *Annali dei Lavori Pubblici*, Roma, Settembre, 1937, p. 717.

in which y_s is the depth of the scour, under the downstream water surface; H_f is the effective drop (see Fig. 12); and q is the discharge per linear unit of the dam. This formula is translated into English units as

$$y_s = 1.32\, H^{0.225}{}_f\, q^{0.54} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2b)$$

in which y_s and H_f are in feet; and q is in cubic feet per second per foot of dam.

In Fig. 12, which is analogous to Fig. 6: $H_f = 1{,}755 - 1{,}692 = 63$ ft; $q = 15{,}000/134 = 112$ cu ft per ft; and, therefore, $y_s = 1.32 \times 63^{0.225} \times 112^{0.54} = 43$ ft. According to Fig. 6, $y_s = 1{,}692 - 1{,}648 = 44$ ft. In other words, Eqs. 2 conform perfectly to the experiences observed in the Brown Canyon model.

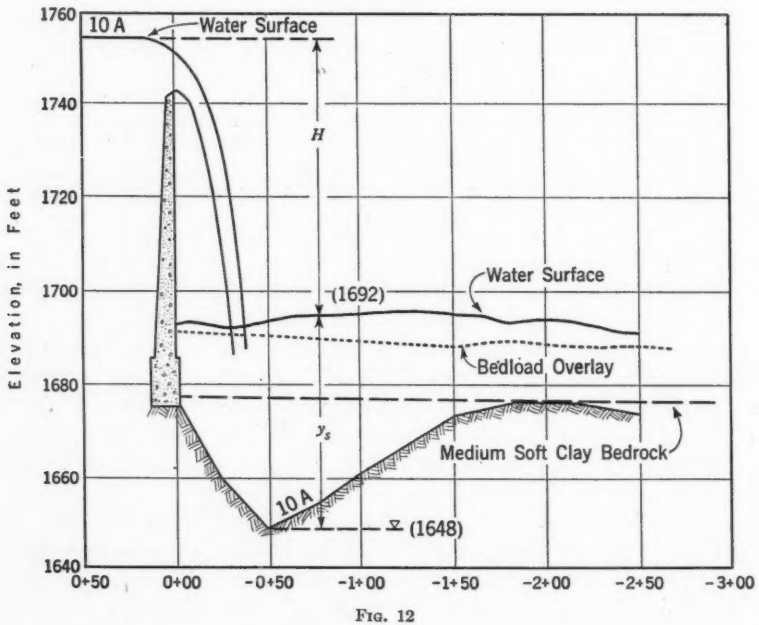

FIG. 12

When formulas such as Eqs. 2 cannot be used for the design of hydraulic structures, at locations where the tailrace is subject to scour, and where it is not possible to reduce the model material to the relative scale of the prototype material, observations to determine the maximum scour, or scouring limit, can be made by another experimental method. This consists in making successive observations on the model with smaller and smaller grain sizes without regard to the grading of the prototype material. Trials must be begun with rather large gravel and then repeated with successively smaller gravel. The scour will be deeper and deeper with each reduction in grain size until a limiting depth is soon reached.

This method has been used by the writer on a model of Brown Canyon Dam (scale 1:50) similar to that used by the authors, constructed in the

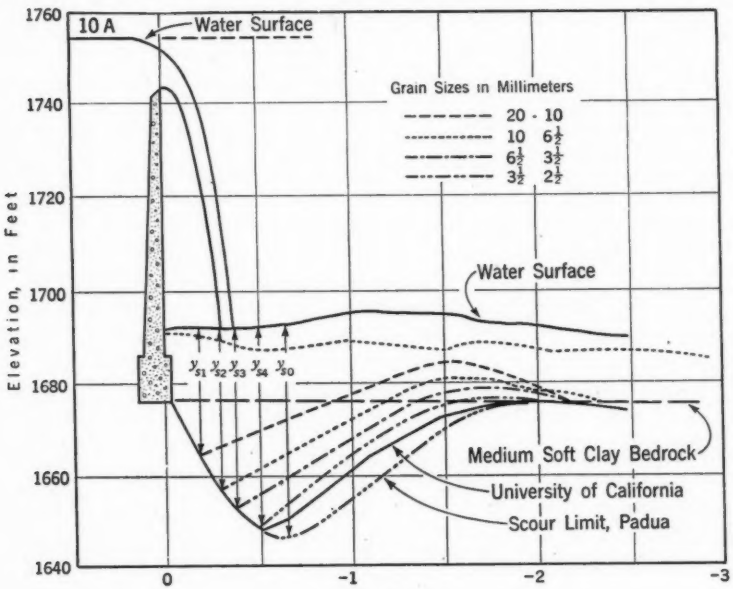

FIG. 13

hydraulic laboratory of the University of Padua in Padua, Italy. The results of these trials are plotted in Figs. 13 and 14. Fig. 14 gives the relation between depth of scour and grain size. At grain size zero the scour limit, $y_{s0} = 47$ ft, is obtained. This value is near enough to that found by the authors (44 ft) so that the difference may be said to be due to errors in reproducing various accessories and parts of the model to an accurate scale ratio.

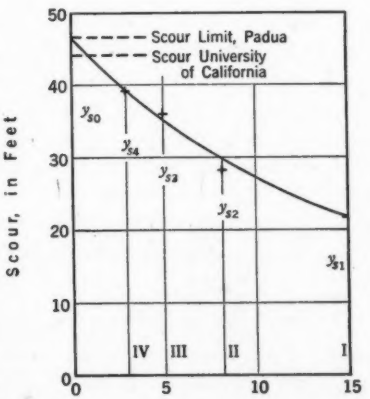

FIG. 14

This method should lead to good results for any type of hydraulic structure, without necessarily conforming to any reduction, in scale, of the prototype material, and of its grading curve (if available). Results of this kind, with some limitations, were described in a previous paper regarding scouring experiences at Conowingo Dam, at Sennar Dam, and at other structures.[8]

[8] "Sulla relazione che intercede fra gli scavi osservati nelle opere idrauliche originali e nei modelli," by E. Scimemi, *L'Energia Elettrica*, Milan, November, 1939.

J. W. JOHNSON,[9] Assoc. M. ASCE.[9a]—A search through published reports on hydraulic model studies shows that investigations have been made on a large number of hydraulic problems. However, the study by the authors of the Arroyo Seco structures appears to be the first comprehensive model investigation on the problem of the movement and deposition of sediment in the vicinity of debris barriers. The advantages of model studies for debris barrier investigations are many; a large variety of factors such as barrier spacings, order of constructing a series of barriers, and sediment loading can be studied in detail. Observations on field installations would require years to yield information of comparable value. The experiments reported by the authors and those made at a later date on the other barriers of the Arroyo Seco system[10,11,12] have permitted the formulation of certain general principles of the action of debris barriers.[13]

The section of the paper concerning the deposition studies is of particular interest to the writer. It was fortunate that the studies on sediment deposition were in the early stages of the model work when the January, 1943, storm occurred on the Arroyo Seco watershed. During, and following, this storm, prototype data were obtained on: (*a*) The total volume and slope of the debris deposited behind the Brown Canyon Barrier, (*b*) a hydrograph of stream flow at a gaging station a short distance downstream, and (*c*) a record of water levels in the basin during the flood. Thus, with the total volume of debris transported by the stream during the flood (except for the volume of very fine suspended matter which passed over the spillway) and the slope of the final deposit being known, the model of the Brown Canyon Barrier was operated by varying the base length of the hydrograph until the volume and slope of the debris deposits in the model corresponded with those observed in the prototype. In all subsequent investigations of the Brown Canyon Barrier and the proposed barriers located farther downstream, the hydrograph and rate of sand feed developed from the data of the January, 1943, storm gave an excellent means of comparing the effectiveness of the various barrier systems, bank protection works, channel correction measures, and other proposed works.

The most important factor in estimating the efficiency of a reservoir as a debris trap appears to be the ratio of original capacity for debris storage to the area of the watershed.[14] From the model studies of the various barrier systems the volume of debris that could be stored in each system was estimated and the capacity-watershed ratios were computed and are summarized in Table 2 (the various systems include the Brown Canyon Barrier, a high barrier

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[9] Associate Prof. of Mech. Eng., Univ. of Calif., Berkeley, Calif.

[9a] Received October 17, 1946.

[10] "Brown Canyon and Weir Debris Basins and Barriers," by W. K. Bermel, Fluid Mechanics Laboratory, Vol. II, Univ. of Calif., Berkeley, August, 1945 (unpublished).

[11] "Report on Low and Intermediate Barriers in the Weir Barrier Reach," by G. R. Loucks, Fluid Mechanics Laboratory, Vol. III, Univ. of Calif., Berkeley, August, 1945 (unpublished).

[12] "Report on Bank Protection," by W. U. Garstka, Fluid Mechanics Laboratory, Vol. IV, Univ. of Calif., Berkeley, May, 1945 (unpublished).

[13] "Movement and Deposition of Sediment in the Vicinity of Debris-Barriers," by J. W. Johnson and W. L. Minaker, *Transactions*, Am. Geophysical Union, 1944, p. 901.

[14] "The Control of Reservoir Silting," by C. B. Brown, *Miscellaneous Publication No. 521*, U.S.D.A., 1943.

at the Weir Site (Fig. 1), and two possible systems of low barriers in reach 4 instead of the high barrier at the Weir Site).

The ratios of capacity versus watershed area (capacity-area ratio) for the various barrier systems are of interest in connection with the relationship between annual storage loss and capacity-area ratio as computed for existing water-supply reservoirs in Southern California, as shown in Fig. 15; the plotting data for this figure being obtained from published data on reservoir silting.[15] Carl B. Brown, Assoc. M. ASCE,[14] states that in reservoirs used for water storage an annual storage loss of one half of 1% is considered serious enough to require measures to control silting. A comparison of the data in Table 2 with Fig. 15 indicates that the Brown Canyon Barrier and the proposed high barrier at the Weir Site might be expected to have an annual loss of storage capacity near this critical rate; however, the 7-ft and 15-ft barriers in reach 4 (see Table 2) definitely would be expected to have excessive silting

TABLE 2.—Ratios of Capacity Versus Watershed Area, Arroyo Seco

(Units Are Acre-Feet per Square Mile)

System	Barriers		Capacity (cu yd)	Capacity-area ratio
	No.	Height (ft)		
Brown Canyon...	1	65	1,140,000	49.4
Weir Site.....	5	7	120,000	5.3
	3	15	340,000	11.2
	1	60	1,528,000	50.0

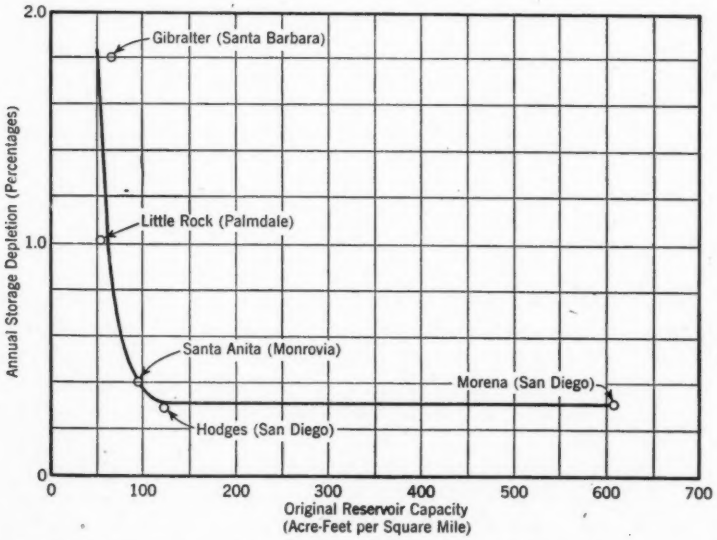

Fig. 15.—Relationship Between Annual Depletion of Reservoir Capacity and Capacity-Watershed-Area Ratio for Various Southern California Reservoirs

[15] "Silting of Reservoir," by H. M. Eakin and Carl B. Brown, *Technical Bulletin No. 524*, U.S.D.A., revised August, 1939.

rates. The model studies, in fact, indicated that the basins in these two low-barrier systems would become almost completely filled during a single flood of the magnitude of that of January, 1943. It is admitted, of course, that the data used in plotting Fig. 15 are too meager to give a definite relationship between annual storage loss and capacity-area ratio; however, the general magnitude of the critical value of the ratio appears reasonable. Future sedimentation surveys of water and debris storage basins in Southern California will provide needed data on reservoir silting rates in that area.

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DISCUSSIONS

RELIEF WELLS FOR DAMS AND LEVEES

Discussion

BY W. A. WALL AND C. A. STONE

W. A. WALL,[11] ESQ., AND C. A. STONE,[12] ESQ.[12a]—An interesting and comprehensive discussion of the theoretical principles of underseepage control and the relief of subsurface pressures by the use of relief wells is presented in this paper. The writers, having been concerned with the theoretical design of relief well systems for a number of projects, have used these design principles, and those presented by Preston T. Bennett,[13] M. ASCE, as the "Modified Muskat-Jervis Well-Spacing Formula." The formulas thus applied give results that are in close agreement; and the Bennett method simplifies, to some extent, the assumptions that must be made to permit a mathematical solution. In both cases, however, assumptions must be made for the well spacing, well radius, and well losses; only by repeated trials can a theoretically ideal combination of these variables be found which will provide the required subsurface pressure relief most economically.

To simplify the design computations and to facilitate the determination of the most economical design of a well system, the writers suggest a well-spacing chart for the design of pressure relief well systems. Fig. 10 is a further modification of the "Modified Muskat-Jervis Well-Spacing Formula." The intent and purpose of this chart is to provide a reasonably accurate method utilizing fewer initial assumptions in determining the optimum well system design. The writers were unable to eliminate all the assumptions; however, those that must be made are of minor significance, and the chart provides a rapid method for solving for the optimum well spacing for a given set of conditions.

In the usual problems of relief well design, particularly where the design is made in conjunction with the design of a levee or a dam, the maximum permissible head midway between wells is the prime factor governing the design.

NOTE.—This paper by T. A. Middlebrooks and William H. Jervis was published in June, 1946, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1946, by Henry C. Barksdale, Williard J. Turnbull, and Glennon Gilboy.

[11] Engr., Office, Dist. Engr., Kansas City Dist., Kansas City, Mo.

[12] Engr., Office, Dist. Engr., Kansas City Dist., Kansas City, Mo.

[12a] Received October 16, 1946.

[13] "Comments on the Design of Relief Wells," by P. T. Bennett, Rept. on Conference on Control of Underseepage, U. S. Waterways Experiment Station, Vicksburg, Miss.

In addition to the head midway between wells, the following factors are known, can be fixed, or must be assumed regardless of the analysis used: (1) The maximum elevation of the impounded water; (2) the maximum allowable pressure in the line of wells; (3) the elevation of the well discharge; (4) the length of path

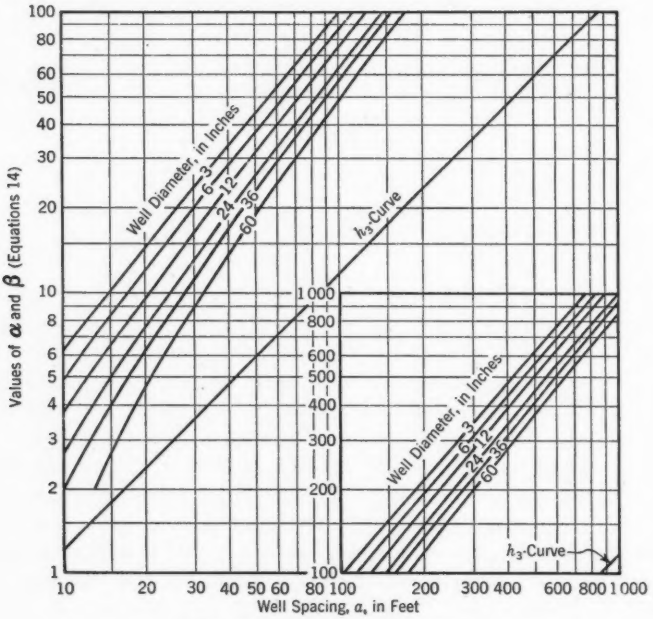

FIG. 10.—WELL-SPACING CHART

of flow from the source to the line of wells; and (5) well losses. The well losses, which are dependent on the quantity of discharge, size, and type of construction, must be assumed. In properly designed wells, these losses (consisting of the well-screen entrance loss, friction loss through screen riser and casing, and velocity head loss), generally, are of a magnitude of less than 1 ft of head. Except in unusual circumstances this value should not be exceeded and can therefore be assumed for the initial computations.

Referring to Fig. 11, let the difference in head h_m between the midpoint of wells and the wells be expressed as

$$h_m = \alpha \frac{h_r}{l} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (14a)$$

or

$$\alpha = \frac{h_m\, l}{h_r} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (14b)$$

Furthermore, let the difference in head h_3 between the midpoint of wells and the average in the plane of wells be expressed as

$$h_3 = \beta \frac{h_r}{l} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (14c)$$

By entering Fig. 10 with value α, obtained by solving Eq 14*b*, the required well spacing a can be determined for the well size selected, and the value of β can be determined from the h_3-curve for that well spacing.

After solving Eq. 14*c* for h_3, the rate of discharge, Q, per well is obtained by the formula:

$$Q = \frac{k\,a\,d}{l}\,(h_r + h_3) \qquad (15)$$

A check is then made to determine the accuracy of the assumption made for the well losses, and corrections are made if required. With a little practice, the value can usually be approximated within a few tenths of a foot.

Generally, a slight increase in the well size may be justified in the judgment of the designer to compensate for the decrease in efficiency of the well over a period of years; but an arbitrary decrease of well spacing to compensate for inaccuracies in the coefficient of permeability cannot be justified, since the well-spacing requirement is not a function of the permeability. The maximum well spacing should not exceed the distance from the source to the line of wells, or l-distance. This is particularly true where large wells are used with a relatively short distance to the source. This is not in conformity with Messrs. Middlebrooks and Jervis who have indicated that the maximum well spacing should not exceed twice that distance.

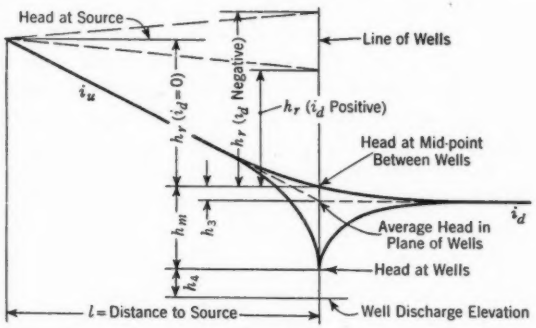

FIG. 11

Fully penetrating wells should be used if economically feasible, as most natural formations present extremely wide variations in permeability from the top to bottom of the stratum. The percentage of penetration of a well should be in proportion to the carrying capacity of the pervious stratum rather than to the thickness of the pervious stratum. Since the permeability usually increases with depth, the economy of the partly penetrating well may be more hypothetical than real. Designs based on partly penetrating wells must, therefore, be very carefully considered on the basis of comprehensive underground data or on actual pumping tests on installed wells.

The theory of pressure relief wells is based on the concept that the well is "a hole with no clogging on the sides." It is apparent that, with present construction methods, an installed well is not 100% efficient. Data pertaining to the relationship of actual production or efficiency of an installed well to the production or efficiency of the theoretical well are very limited or nonexistent. However, since the most open types of well screen have less than 50% open area, it appears that the entrance face area of the well is reduced more than 50% when the well screen is placed in the well to support the formation. Although

the entrance face area may be increased somewhat by surging the well to remove the fines, or by placing a gravel filter around the well screen, it is doubtful whether the effective entrance face area of the installed wells is more than 50% of that of the theoretical well. For this reason it is considered that the installed well diameter should be at least three times greater than the theoretical diameter. Although the excess well diameter will provide for subsequent loss of efficiency from clogging and encrustation, it will also produce more water than is required for pressure relief. In those instances where excess well discharge is of consequence from the standpoint of pumping or conservation, an orifice or valve may be placed in the well for reducing the flow to that actually required to produce the desired pressure reduction.

Since mathematical design formulas for relief well systems are based on idealized assumptions as to the uniformity of the pervious stratum and blanket, and the determination of permeability, which is at best only a good approximation, the results from the formulas should not be considered as an exact design but rather as an approximation to aid the judgment of the designer. With this thought in mind it is the writers' opinion that reasonably accurate results quickly computed are more desirable than mathematically exact solutions achieved through repeated trials. Also the writers wish to emphasize that generally wells of large effective radius are necessary to provide sufficient entrance face face area, and that well screen and riser losses should be kept to a minimum.

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DISCUSSIONS

TRUCK SPEED AND TIME LOSS ON GRADES

Discussion

BY JONATHAN JONES

JONATHAN JONES, M. ASCE.[9b]—The impression a layman receives from reading this paper is that the adverse effect upon general traffic of the slowing down of trucks on ascending grades is not a very serious matter; this impression is summarized in the two concluding sentences of Section 16. The author may be justified in reaching this conclusion for his locality, and for overpasses required in otherwise flat terrain. He considers, however, that his study is more widely applicable than that. In the rapidly succeeding hills and valleys of the state of Pennsylvania, the backing-up effect of trucks upon general traffic on ascending grades is seriously exasperating and, human nature being what it is, a source of great danger.

All may be well on four-lane divided highways, assuming that all slow-moving trucks will occupy the right lane in each direction; but on two-lane and three-lane highways, ascending, faster vehicles cannot pass a truck without extreme danger; nor, after passing the crest, can they often pass because at that stage the trucks will accelerate quickly to the legal limit of speed. Consequently there is no greater boon desired by the general driver in a hilly state, than the elimination of the backing-up effect of slow-moving trucks on steep ascending grades.

The author's statement "Ordinary conditions of the profile and ordinary proportions of truck traffic apparently are not likely to justify the construction of an extra traffic lane * * *" should be taken as applying to overpass grades and not to the means of crossing a natural rise. There is in such situations an alternative means which has taken hold, to what the writer considers an excessive degree; and the intent of these remarks is to express a belief that here the construction of the extra lane, where relief is needed, will usually be superior to the alternative practice of making a deep cut to reduce the grade.

Fig. 7 will serve to illustrate this point in terms of a highway practice of which the writer is well aware through observing the construction of improved highways in his vicinity. This diagram shows roughly the frontal view and the

NOTE.—This paper by J. W. Stevens was published in September, 1946, *Proceedings*.

[9b] Received October 1, 1946.

plan view of an incline from a valley to a crest, on which it may be assumed that the natural grade would be severe, and that this may be reduced to half or less by making the deep cut indicated as scheme A. So doing does not, however, enable all trucks to climb the grade without reduction of speed, but only some of them. The others reduce their speed somewhat, although not so much as on the original grade. The delay to following traffic is therefore somewhat mitigated; the danger is not removed. Passing on the center lane is dangerous and in many instances forbidden by law. The cost of the cut is extreme; in one observed instance it added at least three months to the construction time, during which traffic was detoured, because of the necessity for blasting solid rock.

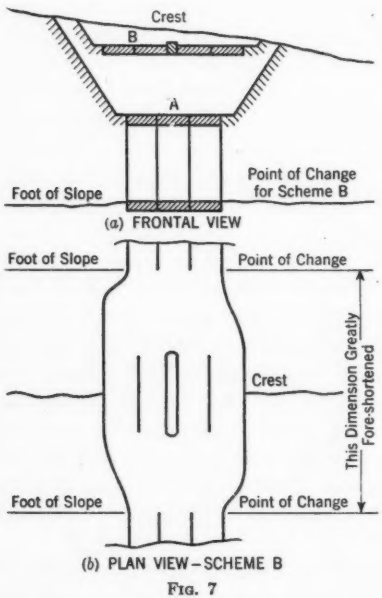

FIG. 7

If, however, this road were constructed from valley to crest as shown as scheme B, and in the plan view, Fig. 7(*b*), providing an additional lane on the right side of the ascending pair, and separating the ascending and descending pairs with a raised element, then all passenger and light truck traffic could negotiate the steep grade in high gear, without any danger from oncoming traffic, and therefore without the need for special sight distance. The cost (because the ground would be only barely scratched instead of deeply cut) should be little more, and frequently would be less, then alternative A. The required right of way would be reduced. The pleasure of the average passenger would be enhanced, because he would be viewing a landscape instead of the side of a cut. Trucks on the right-hand lane would naturally be traveling at considerably slower speed by scheme B than by scheme A; but their loss in this respect would be the gain of all others.

As a user of the highway both for business and pleasure, the writer believes that the construction of an extra traffic lane over natural rises is very frequently justified, and that the fullest consideration should be given to providing this lane in accordance with scheme B rather than seeking to avoid it by the adoption of scheme A.

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DISCUSSIONS

ANALYSIS OF UNSYMMETRICAL BEAMS BY THE METHOD OF SEGMENTS

Discussion

BY SOL LIFSITZ

SOL LIFSITZ,[14] ESQ.[14a]—By clarifying some of the "hard-to-read" paragraphs of the paper Mr. Conwell has supplemented the paper considerably. As to the omission of the derivation for Eq. 2 which Mr. Conwell mentions, the writer was of the opinion that its inclusion would serve no useful purpose. The demonstration given by Mr. Conwell, based on the principle of work, is only one of several proofs available in the literature on moment distribution. As a matter of fact, the proof of Eq. 2 is inherent in Eqs. 19, where it is seen by inspection that $C_{AB} K_{AB} = C_{BA} K_{BA}$.

Mr. Hutchinson finds the work involved in the computation of fixed-end moments by the method of segments disappointing, citing the method of "multiplying factors" developed by Carl Wagner, Jr., Jun..ASCE, as an alternative for comparison. The criterion for deciding on either of two methods, both yielding the same degree of accuracy, should be the ease of computation and the time required to obtain results. After comparing the two methods, the writer finds that with the same accuracy of results the time required by his own method is considerably less than that required for the suggested alternative.

Mr. Revelise is quite right in stating that the application of the method is limited to members in which the variation in moment of inertia is a continuous function, or at most contains but a single discontinuity in each segment of the beam. The efficient use of the method is confined to cases for which tables or curves of the segmental beam constants are available, and to the writer's knowledge such data for beams of highly discontinuous outline do not exist. For cases in which it is necessary to derive the beam constants from first principles, there is no economy of time over other methods, of which Mr. Revelise has given one very good example.

NOTE.—This paper by Sol Lifsitz was published in March, 1946, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1946, by William A. Conwell, Ralph W. Hutchinson, and Thomas P. Revelise; and November, 1946, by Victor R. Bergman, and A. A. Eremin.

[14] Civ. Engr., Board of County Road Commissioners, Wayne County, Detroit, Mich.

[14a] Received November 6, 1946.

Mr. Bergman's method for finding stiffness and carry-over factors appears to be very simple and bears evidence that his approach to the problem in his method of sections closely parallels that of the writer's method of segments.

In conclusion, the writer wishes to thank the discussers for their interest in the paper and their kind and helpful criticism, which he found very stimulating.

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DISCUSSIONS

SOME THOUGHTS ON ENGINEERING EDUCATION

Discussion

BY WILLIAM A. CONWELL, AND A. AMIRIKIAN

WILLIAM A. CONWELL,[33] M. ASCE.[33a]—The kind of material about which practicing engineers often talk but seldom write is contained in this paper. The topics, of course, have been treated at length by engineering educators in the rôle of scholars discussing their specialities. Such dissertations lack the freshness and exhilaration which are likely to characterize the probings of an intelligent man in a field in which, although he is not a specialist, he is highly interested. Such are the characteristics of this paper. The thoughts and musings are the type which might be heard in an informal discussion of the subject by engineers. In this instance, however, the ideas are supplemented by well-arranged data which give them force and weight ordinarily absent in informal discussion.

It is interesting to examine the proposed revisions to the methods of engineering education in the light of the functions of those concerned—the individual, the school, and the employer.

Under the system generally employed prior to World War II, the individual followed a course of instruction which was largely technical. He was left principally to his own resources in developing the nontechnical requirements of his profession, and of his cultural background. The school outlined courses and taught subjects which were technical or which barely skirted the fringes of the nontechnical requirements. Employers did one of two things, depending upon their needs. With the confidence that the logical thought processes induced by an engineering education would be of value to him whatever his business, one type of employer hired graduating engineers regardless of their specialties; acquainted them with his organization by a course of indoctrination; and then used them to solve his problems, whether in the field of engineer-

NOTE.—This paper by Donald M. Baker was published in April, 1946, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1946, by E. S. Boalich, Russell C. Brinker, I. Oesterblom, Samuel T. Carpenter, Lynn Perry, and L. E. Grinter; and October, 1946, by Robert O. Thomas, Clement C. Williams, N. W. Dougherty, H. A. Wagner, M. E. McIver, and Scott B. Lilly.

[33] Gen. Engr., Structural Eng. and Design Dept., Duquesne Light Co., Pittsburgh, Pa.

[33a] Received September 30, 1946.

ing or not. It was this type of employer who voiced opposition to excessive specialization according to the 1940 analysis of the American Association of Engineers.[2] The other type of employer hired the graduate as a technologist and put him to work immediately on the solution of engineering problems. In the latter instance, the engineer's experience, gained in the solution of practice problems as a student, was indispensable.

Under the plan of education proposed by the paper the individual would pursue a more general course and thus not be required to devote as much of his extracurricular activities to self-development. The school would offer the general course stressing fundamentals in technical subjects, eliminating practice problems, and including considerable nontechnical and probably cultural matter. Needless to say, the school would have to "sell" such a course to the embryo engineers who normally look askance at any subject that lacks the quality of being immediately and practically applicable in professional practice. The school would further expand opportunities for the employed graduate to continue his studies, a commendable proposal under any system of education, past, present, or future. If the employer is of the first type, interested in men trained as engineers regardless of branch, the suggested plan suits him admirably. All engineering graduates would be eligible for his indoctrination course. The second type of employer, who wants a technologist whom he can put to work on engineering problems, will find no graduate with the requisite experience in practical problem solution. He, too, must institute some kind of training. To borrow from Hardy Cross, M. ASCE, he must take the young engineer who has "hitched his wagon to a star" and induce him to get "his feet on the ground."[34]

In this comparison of the former and the proposed methods of education, one can discern the definite shift of responsibility from the individual to the school, and from the school to the employer. The school takes over the student's former responsibilities in the nontechnical field, and the employer, unless he confines his employees to men with advanced degrees, assumes the responsibility of the school for training in practice problems. This trend may be thought of as consistent with that in many other fields and as accompanying the higher degree of socialization which we shall continually find in our civilization; but it could hardly be said to enhance the engineer's individuality, at present one of his most precious possessions.

The writer is in hearty agreement with the author's premise that, in his undergraduate training, the student should be imbued with a sense of the importance of professional progress after graduation. Real progress is demonstrated, of course, by the engineer's performance of increasingly difficult work as his career develops; but this progress is often reflected in, and enhanced by, activities in three fields—advanced degrees, registration, and advanced grades of membership in engineering societies.

Appreciation of the value of the master's and the doctor's degrees is mounting as time goes on and there can be little doubt of their eventual attainment of

[2] *Professional Engineer*, November, 1940.
[34] *Transactions*, ASCE, Vol. 96, p. 140.

the respect due them in the engineering world. The same cannot be said of the professional degree which is sought by relatively few graduates. The attitude toward this degree is surprising in view of the fact that its essence is recognition of experience, a most valuable asset of the practicing engineer.

The trend in the field of registration for license to practice is "all to the good." The time should not be far off when all professional engineers will recognize the full value of registration.

Appreciation of membership and advanced grades in engineering societies is lacking in many engineers. As against the author's opinion, it is the writer's firm conviction that failure of an engineer to obtain membership in the engineering societies and to advance his grade consistently lies in his apathy rather than in his not having had opportunity to perform work which would entitle him to recognition.

It is the writer's belief, contrary to that of the author, that the economic depression of the 1930's had a profound effect upon the stability of Junior Members of that period.

There is some question as to whether an anomaly is not presented by the author's proposed revision of courses which excludes some subjects and adds others. One of the author's arguments for elimination of practice problems is that the graduate must wait eight or ten years before he has an opportunity to apply what he has learned (a premise with which the writer cannot wholly agree). At the same time the author recommends courses in "men," "money," and "management" for inclusion in undergraduate curricula; but in tracing the professional progress of the engineer, the author envisages his dealing first with "matter," next with "men," then with "money," and finally, at the height of his career, with "management." If the reason for eliminating practice problems is valid, it would seem that a similar reason would hold for eliminating the last three "M's" from the undergraduate work. These thoughts are presented in the belief that the author, in his closing discussion, might like to clarify an apparent inconsistency. They are not meant to imply that courses, particularly those dealing with "men," are out of place before graduation. Ability to get along with one's fellowmen is of paramount importance; and, if a student's home or early school training has failed to develop him in this direction, something should be done about it in college.

The writer questions how effective a five-year or ten-year forecast of the market for engineers can be. It is unlikely, for instance, that the great demand for radio engineers in the 1920's could have been predicted. On what basis could there have been foretold the need in the early 1940's for army engineers with training in "men" and "management" far beyond their years and experience? Flexibility in training, the second alternative suggested by the author, would seem to be the solution to this problem; but the goal of flexibility does not necessarily (in the opinion of the writer) require the elimination of practice problems in undergraduate work.

If the prewar graduate was "primarily a trained, rather than an educated person," his successor, under the proposed revisions, would probably continue to be so. Although certain technical training would be eliminated and more emphasis would be placed on purely educational subjects, the suggested courses

in speaking, writing, business, etc., would be in the field of training rather than education.

A. AMIRIKIAN,[35] M. ASCE.[35a]—If the success of the present-day engineering education is to be gaged by the success of its product, as suggested by the author, the statistical data presented in the paper do not reflect a favorable condition in so far as the great majority of the members of the profession is concerned. This is unfortunately true whether success is measured by the rate of the young engineer's progress in the profession, his attained standing and prestige in the community, the recognition of his contribution, or the compensation for his services.

Causes contributory to this unhappy situation are many and complex. Although some of these causes may be traceable to certain deficiencies in engineering education and training, most of them in reality are related to engineering practice. As observed by the author, engineering is no longer practiced as a free profession, but is conducted along lines similar to a business enterprise. The change is not new; it has been taking place during the past several decades. As a result, the individual practitioner, the true representative of the free profession, is practically nonexistent today. He is forced either to seek employment with large engineering and architectural organizations or to join forces with other engineers to form an engineering firm, and thus become either an employer or an employee.

Various reasons are advanced for the evolution of engineering practice from a free profession to a competitive business enterprise. According to the opinion of some, this change is due to modern trends of industrial development with corresponding growth in engineering projects requiring the services of large engineering organizations for their execution. Others think it is due to a keen competition predicated on present-day business methods and practices. Regardless of what the real reasons may have been to cause this change, the important fact is that it has occurred. From an educational viewpoint, the only thing that can be done to improve the lot of the young engineer is to give him a training adaptable to the requirements of current practice.

An examination of engineering curricula will reveal that during the first half of the twentieth century no significant changes have been made in either programs of instruction or methods of training. The institutions are still engaged in the task of producing the finished product—the professional engineer. In this work, efforts are being made to equip the candidate with the necessary knowledge and training to practice engineering in the same way that students are being trained to practice medicine and law. At first approach, the program may appear as proper and its objective as logical; but a closer study of the means utilized in the program and the problem of professional practice of the finished product will bring out serious doubts concerning both the adequacy of the means as well as the propriety of the objective.

The first point of the argument needs little elaboration. It is generally admitted that a four-year engineering course is too short a period for the re-

[35] Head Engr., Bureau of Yards and Docks, Navy Dept., Washington, D. C.

[35a] Received October 31, 1946.

quired training. The disparity between the set objective and the allocated time will be more apparent when comparison is made of the seven-year to eight-year training period prescribed by the legal and medical professions. The inadequacy of the means is further illustrated by the fact that there are few, indeed, if any, institutions which are equipped to give the specialized training required by present-day engineering practice.

The other considerations concern the practical problem of placement of the finished product. Since the young engineer cannot open an office and practice his profession, as would a lawyer and a physician, after graduation there still remains the difficult task of securing a proper position. Strange as it may seem, his specialized training—even when assuming that he was able to obtain it—not only would be of little help in securing employment, but, in most cases, would prove a handicap. This is due to the fact that a young graduate is seldom able to find a position that will correspond to his particular specialization in school. Under normal circumstances, he has but little choice for employment, and usually he accepts the first job that comes along. Furthermore, because of his lack of experience, the organization that offers him employment considers him as a mere recruit who must go through the "grind"—regardless of his specialization or particular qualifications. Stated more frankly, his acceptance into the profession does not materially differ from that of an apprentice into a trade. The course of his later progress is equally discouraging—even more so than that of an apprentice. As a result, after surviving his initial disappointment, the young engineer keeps on changing his place of employment at frequent intervals, trying to adapt himself to the conditions, and, finally, he either becomes reconciled with the situation or he leaves the profession.

Unfortunately, little attention is focused on the latter phase of the problem. The statistical data presented in the paper concerning change in employment and abandonment of the profession reveal an alarming condition. According to the data, the turnover runs as high as 60%, and as many as 50% eventually leave the profession. Essentially, this is a condition suggestive of both maladjustment and malcontentment. Obviously, no changes in the educational system can appreciably affect the latter factor, since it is due entirely to current methods of engineering practice. The only thing that engineering institutions can do in this respect is to take the glamour out of engineering education and present a clear and realistic picture of the profession. Such a warning will, at least partly, obviate future disappointments and disillusionments on the part of the student. On the other hand, conditions of maladjustment can be greatly improved by introducing certain necessary changes in the educational system to provide the needed flexibility of adaptation. Briefly, these changes would consist of the following:

(*a*) *Undergraduate Work.*—Confine undergraduate instruction to courses in liberal arts and general engineering only. Assuming a four-year curriculum, the first two years would be devoted to liberal arts and the latter two years to engineering. The courses in the former period would consist primarily of social and cultural subjects; in the latter period, the main subjects would consist of three important "Ms" which form the backbone of any branch of engineering—

namely, mathematics, mechanics, and materials. There would be no specialized courses, and only simple and elementary applications of the fundamentals to a particular branch of engineering.

The advantages of such a combined undergraduate course are many. From the administrative standpoint, optimum use would be made of the institution's facilities and staff, and there would be little conflict or confusion in the arrangement of the curricula. From the point of view of the student, the derived benefits would be still more important: (1) His liberal education would provide a cultural background that would aid him in later life regardless of what line of endeavor he might eventually follow; (2) there would be ample time for a more considerate choice of a profession and an opportunity for a desired change at relatively later stages of instruction; and (3) the placement and adjustment problems after graduation would be greatly facilitated because of the general nature of the training.

(*b*) *Graduate Work.*—Confine postgraduate work to specialized courses in various branches and subbranches of engineering. The main purpose of these courses would be to provide up-to-date professional training required in current engineering practice. To be eligible for admission, the candidate should have completed undergraduate training and, in addition, should have had a minimum of five years of engineering experience. The importance of the latter requirement cannot be overemphasized. During such a trial period the young engineer would have had a good taste of the methods and practice of the profession for making his final decision; also, he probably would have located the particular line of engineering for which he is best suited.

The courses may be arranged on the basis of both resident and nonresident instruction. In most cases, however, the latter arrangement would prove to be the most satisfactory, since it would assure the practicability of the system. The duration of the period may vary from one to three years, in accordance with the requirements and the complexity of the specialization, and the training would culminate with the granting of a professional degree. As for the necessary instruction staff, the faculty would consist mostly of practicing engineers engaged on a part-time basis—leading specialists in the various fields of engineering. In this connection, it is to be noted that, unlike the medical and law schools, little or no attempts are made by engineering schools to utilize such talent.

(*c*) *Research Work.*—Supplement graduate courses with research work. There are many problems in the various fields of engineering which need elaborate research and experimentation. Most of these problems are directly related to the work of the practicing engineer. As a matter of fact, such problems constantly arise in devising new techniques to make more advantageous use of materials, or to improve the methods of applying an existing technique. If satisfactory arrangements could be made by the engineering schools, it would be possible for the research-minded engineer to bring his problems to the institution and work out a solution by utilizing its aid and facilities. Engineering research conducted on this basis would yield more practical and valuable results than are obtained from the present system of graduate work by which research is incidental and serves primarily as a vehicle for the inexperienced engineering graduate in his effort to obtain an advanced degree.

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DISCUSSIONS

EFFECT OF STRESS DISTRIBUTION ON YIELD POINTS

Discussion

BY J. F. BAKER AND J. W. RODERICK

J. F. BAKER,[9] ASSOC. M. ASCE, AND J. W. RODERICK,[10] ESQ.[10a]—The main contention in this paper is that the upper yield stress for mild steel subjected to nonuniform stress distributions is much higher than that for an axially loaded tension specimen. Mr. Peterson has conducted tests in pure bending and eccentric tension in which the first signs of yielding, as indicated by the appearance of Lueders lines, represent stresses much in excess of the upper yield stress recorded in the usual tension test. This result is contrary to the conclusions reached by A. Robertson (54),[10b] G. Cook (55), H. Quinney (56), and J. L. M. Morrison (46), all of whom have shown that, in a tension test on annealed mild steel, where adequate precautions are taken to insure axial loading and where the specimen is tested in a machine designed to record upper and lower yield points, the upper yield stress observed is frequently more than 40% greater than the lower value—showing good agreement with the results of tests in pure bending. As confirmation, the diagram shown in Fig. 20 for a specimen of black mild steel was obtained on a Quinney autographic testing machine (56) in the Engineering Laboratory at Cambridge,

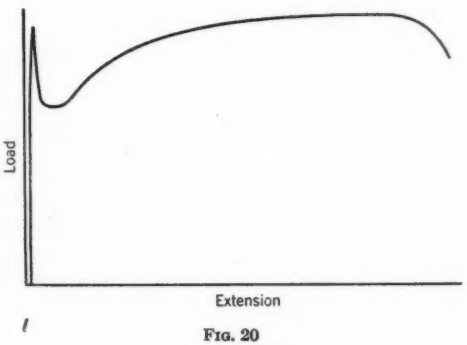

FIG. 20

NOTE.—This paper by F. G. Eric Peterson was published in April, 1946, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1946, by F. P. Shearwood, Edwin H. Gaylord, and L. J. Mensch; and November, 1946, by George Winter, and O. Sidebottom and T. J. Dolan.

[9] Prof. of Mech. Science, Univ. of Cambridge, Eng. Laboratory, Cambridge, England.

[10] Asst. Director of Research, Univ. of Cambridge, Cambridge, England.

[10a] Received October 14, 1946.

[10b] Numerals in parentheses, thus: (54), refer to corresponding items in the Bibliography (see Appendix of the paper), and at the end of discussion in this issue.

England. The steel had received no particular heat treatment and was in the condition "as received"; but despite this the upper yield point was 44% higher than the lower value.

However, in testing a number of tension specimens cut from any carefully heat treated bar, it will usually be found that, whereas the lower yield stresses are remarkably constant, the upper values will often vary considerably, with no evidence of it at all in some specimens. As Mr. Morrison (46) has shown, however, by using a number of extensometers around the circumference of the tension specimen, even the slightest eccentricity can mask the true value of the upper yield stress. When it is realized that these eccentricities can arise not only from the mechanical shortcomings of the loading gear but also from slight variations in properties across the section of the specimen, inconsistent values of upper yield stress as recorded by over-all extension are not unexpected.

It would appear from the stress-strain diagrams in Fig. 10 that no special precautions were taken to insure axial loading in the tension test and also, since no reference is made to the type of testing machine employed, that the lower yield point was presumably obtained by rebalancing the normal lever type machine. If this were the case, it is very doubtful whether either the upper or lower yield points recorded represent the true properties of the material. The load-deflection curve for the beam test is compared with curves based on what are described as the "old theory" and the "new theory" of plastic failure, neither of which give good agreement with the beam test results. In England the former theory is now generally accepted, although where the material has an upper yield point the stress distribution for a partially plastic section of a member subjected to bending is taken to be of the form shown in Fig. 21 as opposed to that given in Fig. 1(*d*). The distribution in Fig. 21 is that established by Messrs. Robertson and Cook (54). The "new theory" referred to by the author is that stated by W. Kuntze (3) which, while it gives reasonable results in isolated cases, cannot be claimed to have any true fundamental basis. One cannot but agree with the criticism of this and other theories offered by F. Rinagl (57):

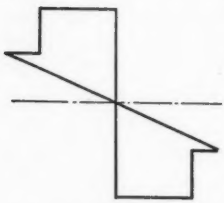

FIG. 21

> "Kuntze's theory, whereby yielding in the extreme fibres is supposed to set in only when the tensile yield limit has been attained in a certain layer—the so called centre of resistance—in the interior of the beam, cannot be accepted either. The 'centre of resistance' is a purely geometrical illusion and has no physical significance. The approximate coincidence with the results of Thum and Wunderlich's tests is only incidental and vanishes as soon as the tests are properly interpreted."

Most continental writers think of W. Prager's theory (12), which assumes that failure of a member subjected to pure bending is instantaneous upon the attainment of the upper yield stress at the extreme fiber, as the "new theory."

The writers have made considerable use of the "old theory" in studying the behavior of rigid welded steel frames tested to collapse (36)(45)(58)(59).

At the beginning of this investigation tests were made on beams to verify the theory; but, although the results obtained were satisfactory, it was realized that better agreement between the yield points in tension and bending might have been obtained if more attention had been paid to rate of loading. Another series of beam tests is in progress taking due account of this factor, and a typical load deflection curve is shown in Fig. 22. When the upper yield stress is neglected as in the version of the "old theory" referred to by Mr. Peterson, there

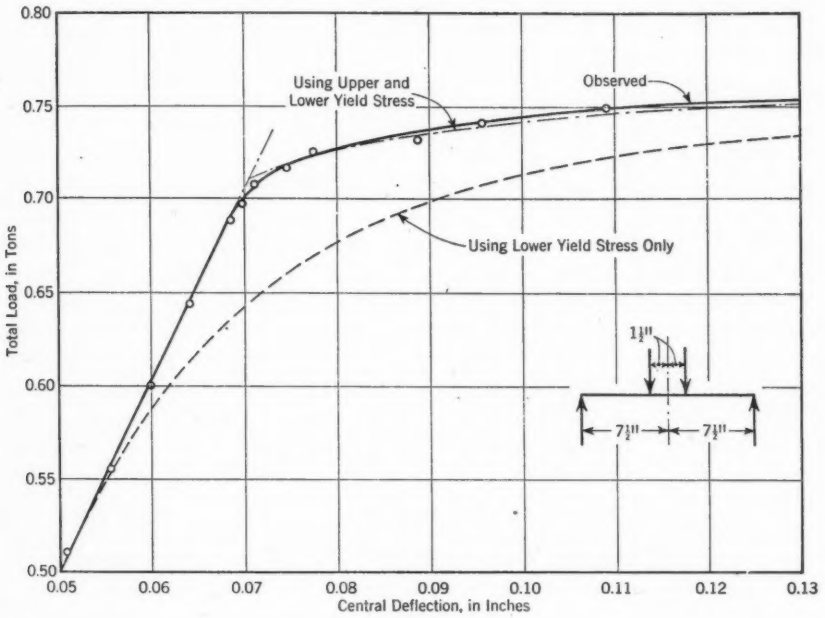

FIG. 22

is considerable difference between the observed and calculated load deflection curves. On the other hand, using the stress distribution of Fig. 21 and an upper yield stress corresponding to the load at which the observed load deflection curve deviates from the straight line, exceedingly good agreement is obtained. The yield stresses determined from the beam test were: Lower yield stress, 13.46 tons per sq in. and upper yield stress, 19.05 tons per sq in. The average lower yield stress from a number of tension tests was 13.84 tons per sq in.

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(46) **"The Yield of Mild Steel with Particular Reference to the Effect of Size of Specimen,"** by J. L. M. Morrison, *Journal and Proceedings*, Inst. of Mech. Eng., Vol. 142, No. 3, 1939.

(54) **"Transition from the Elastic to the Plastic State in Mild Steel,"** by A. Robertson and G. Cook, *Proceedings*, Royal Soc. of London, Series A, Vol. 88, 1913, p. 462.

(55) **"Some Factors Affecting the Yield Point in Mild Steel,"** by G. Cook, Inst. of Engrs. and Shipbuilders in Scotland, February, 1938.

(56) **"Further Tests on the Effect of Time on Testing,"** by H. Quinney, *Engineer*, Vol. 161, 1936, p. 669.

(57) **"Yield Limits and Characteristic Deflection Lines,"** by F. Rinagl, Preliminary Publication, 2d Cong., International Assn. for Bridge and Structural Eng., Berlin, 1936.

(58) **"The Behaviour of Stanchions Bent in Single Curvature,"** by J. F. Baker and J. W. Roderick, 3d Interim Rept., *Transactions*, Inst. of Welding, Vol. 5, No. 3, 1942.

(59) **"The Behaviour of Stanchions Bent in Single Curvature,"** by J. F. Baker and J. W. Roderick, 4th Interim Rept., Research Rept., British Welding Research Assn., 1944.

Correction for *Transactions:* In November, 1946, *Proceedings*, page 1273, Fig. 13 is to be replaced by the following illustration:

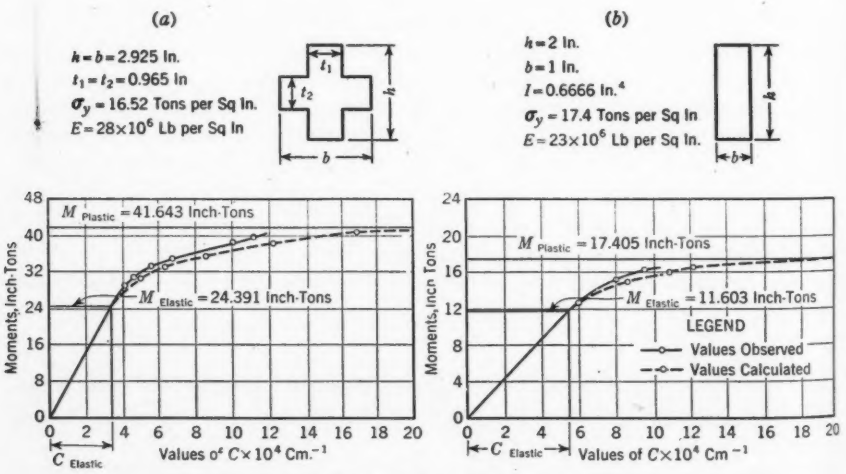

(a)
h = b = 2.925 In.
t1 = t2 = 0.965 In
σy = 16.52 Tons per Sq In.
E = 28×10^6 Lb per Sq In
(b)
h = 2 In.
b = 1 In.
I = 0.6666 In.^4
σy = 17.4 Tons per Sq In
E = 23×10^6 Lb per Sq In.
M Plastic = 41.643 Inch-Tons
M Elastic = 24.391 Inch-Tons
Moments, Inch-Tons
C Elastic
Values of C×10^4 Cm.^-1
M Plastic = 17.405 Inch-Tons
M Elastic = 11.603 Inch-Tons
LEGEND
Values Observed
Values Calculated

FIG. 13.—LOAD-CURVATURE DIAGRAMS FOR TWO STEEL BEAMS ACCORDING TO E. VOLTERRA

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

STRENGTH OF BEAMS AS DETERMINED BY LATERAL BUCKLING

Discussion

BY GEORGE WINTER, AND DAVID B. HALL

GEORGE WINTER,[18] M. ASCE.[18a]—It has been shown repeatedly and convincingly that the generally accepted slenderness ratio for unsupported beams, l/b, is by no means a real criterion for their lateral strength.[19] The great merit of this paper is that the author has succeeded in developing a sufficiently simple set of formulas for inclusion in design specifications, all depending on the compact parameter $\frac{l\,d}{b\,t}$. These formulas (Eqs. 6 to 8), however, are not as general as may appear from the paper, and sometimes they result in greatly misleading and uneconomical values. Furthermore, no rational justification is given in the paper for this particular parameter, except that it happens to fit rather satisfactorily the stress magnitudes obtained from the exact formulas.

It is the intent of this discussion to show in a rigorous manner: (*a*) Why it is this particular parameter and no other which gives satisfactory results for the author's beams; and (*b*) the limitations of the proposed formulas. Finally, for beams to which Mr. de Vries' equations are found not to be applicable, another simple formula is proposed to supplement those given in the paper.

Expressions for critical loads and stresses of unsupported beams have been published before. In particular, the writer,[20] in collaboration with the late Lt. R. K. Schrader,[21] Jun. ASCE, published in 1941 a set of expressions for critical loads and moments, which not only covers Mr. de Vries' cases but is more general and also deals with additional types of loading. The only difference in Mr. de Vries' treatment is that he accounts for the second term in the Fourier expansion of the elastic curve, whereas the writer was satisfied with the first term. The negligible influence of this second term can be noted not only

NOTE.—This paper by Karl de Vries was published in September, 1946, *Proceedings*.

[18] Associate Prof. of Civ. Eng., Cornell Univ., Ithaca, N. Y.

[18a] Received September 27, 1946.

[19] "Strength of Slender Beams," by George Winter, *Transactions*, ASCE, Vol. 109, 1944, p. 1321.

[20] "Lateral Stability of Unsymmetrical I-Beams and Trusses in Bending," by George Winter, *ibid.*, Vol. 108, 1943, p. 247.

[21] *Ibid.*, p. 261.

in the close agreement of the values in both papers, but also in the fact that the values for y in Table 7 are all rather small as compared with unity.

In a subsequent paper the writer gave the following formula for the buckling stress of such beams:

$$f = \frac{E\,\pi^2}{2\left(\frac{l}{d}\right)^2}\sqrt{\left(\frac{I_y}{2\,I_x}\right)^2 + \frac{K\,I_y}{2(1+\nu)\,I^2{}_x}\left(\frac{l}{\pi\,d}\right)^2} \quad \ldots\ldots\ldots (51)$$

in which ν is Poisson's ratio and K is the torsional constant of the beams.[22] He also showed that Eq. 51, derived for pure bending, can be applied satisfactorily to most practical cases of transverse loading[23]—a finding which is identical with Mr. de Vries' analysis of various loading conditions.

The reason for, and the limitations of, the author's parameter, $\frac{l\,d}{b\,t}$, can be derived rigorously from Eq. 51. The buckling stress depends on the two quantities appearing under the radical. It so happens, however, that for most rolled and built-up sections the first term is negligibly small as compared with the second. For example, for the three I-beams (24I120, 3I5.7, and 12I55), with $l/d = 30$, the values of these two terms are, respectively, 0.00020 and 0.0037, 0.0085 and 0.068, and 0.00073 and 0.009. Therefore, in a practical design formula, the first term in Eq. 51 is negligible for such beams, particularly since both are included under the radical. Neglecting this term, the following simplification can be introduced:

In view of the usual shape of rolled and built-up I-beams, the web contributes very little to I_x, I_y, and K. Consequently, the following approximate expressions can be used for these three quantities—

$$I_x = 2\,b\,t\left(\frac{d}{2}\right)^2; \qquad I_y = \frac{2\,t\,b^3}{12}; \qquad \text{and} \quad K = \tfrac{2}{3}\,b\,t^3 \ldots\ldots\ldots (52)$$

Substitution of these values in Eq. 51, with the first term under the radical equal to zero, results in

$$f = \frac{19{,}500{,}000}{\frac{l\,d}{b\,t}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (53)$$

Eq. 53 differs from Eq. 7 by only 2.5%.

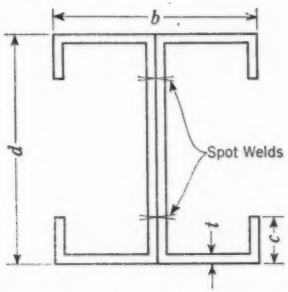
It is thereby established that Mr. de Vries' approximate formula is rigorously justified provided that the first term under the radical in Eq. 51 is negligible as compared with the second term. This condition will be true for all beams whose flange thickness is not too small as compared to the width (to about $b/t = 20$). For beams with still thinner flanges the torsional constant K (which is proportional to t^3) decreases rapidly to such an extent that the second of these two terms approaches the order of magnitude of the first. For example (to consider an extreme case) for 6WF15.5, with $l = 20$ ft, the magni-

[22] "Strength of Slender Beams," by George Winter, *Transactions*, ASCE, Vol. 109, 1944, p. 1334, Eq. 41.

[23] *Ibid.*, pp. 1339 and 1340.

tude of these two terms, in order, is 0.030 and 0.071. Although the first term is still seen to be smaller than the second, it is no longer of negligible magnitude. Indeed, the buckling stress from Eq. 51, for $l = 20$ ft, is 27,000 lb per sq in., whereas Eq. 7 results in only 21,800 lb per sq in. Beams of this character, however, are used infrequently except for relatively small structures with light loading.

However, for structures of this kind new types of sections are coming into increasingly widespread use. Such members are fabricated from sheet steel by cold-forming and, usually, by spot-welding.[24] Sections of this type are manufactured at present by a number of specialized firms in that field and, in the near future, will become available commercially on a wide scale. Sheet steel I-beams are frequently made up from two C-channels spot-welded back to back as shown in Fig. 9. In such beams the thickness-width ratio of the flange is ordinarily of the order of $b/t =$ from 25 to 100. It must be expected, therefore, that in such beams the first of the two terms in Eq. 51 is no longer of negligible magnitude. Indeed, for the two beams whose dimensions are given in Fig. 9, with $l/d = 30$, the values of the two terms, in order, are 0.019 and 0.0034 and 0.00049 and 0.00018. The situation here is the reverse of that found for rolled sections; and, consequently, Eq. 7, if applied to these beams, must result in stresses which err very much on the low (conservative) side.



Beam	t	d	b	c
A	0.075	6.0	5.0	0.75
B	0.050	8.0	3.0	0.60

FIG. 9—LIGHT-GAGE STEEL BEAMS

Since the simple parameter $\frac{l\,d}{b\,t}$ cannot be used for such light-gage I-beams, it will naturally be asked how lateral strength should be determined in the design of sections of this kind. For this purpose Eq. 51 can be used as it is. Most designers, however, will consider it too cumbersome for everyday use. To arrive at a sufficiently simple expression, essentially the same kind of simplification can be used as was employed for rolled sections. It will be noted from the foregoing data that, for light-gage beams, the first term under the radical in Eq. 51 is several times larger than the second. It appears, therefore, that a formula which discards this second term, gives a result not too different from the accurate value of Eq. 51.

(The situation for beam A (Fig. 9) is much more favorable in this respect than that for beam B. However, sections of such extreme shape as beam B will scarcely be suitable for beams, since the low ratio of b/d makes such use very uneconomical. Beam B was chosen as an extreme case to show the maximum discrepancies to be expected from the formula that follows, whereas beam A is probably rather close to average dimension of practical beams.)

[24] "Light-Gage Steel for Peacetime Building," by Milton Male, *Engineering News-Record*, October 18, 1945, p. 525.

For light-gage beams, then, if the second term of Eq. 51 is discarded, the simple formula for the buckling stress is obtained:

$$f = \frac{74{,}000{,}000}{\left(\frac{l}{d}\right)^2} \frac{I_y}{I_x} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (54a)$$

All quantities in Eq. 54*a* are those usually tabulated for commercial sections.

For the two beams in Fig. 9, for $l/d = 30$, the following values of the buckling stress are obtained:

Eq.	Beam A	Beam B
51	24,600	4,300
7	6,900	1,900
54	22,500	3,700

It is clear that the author's values cannot be used for such light-gage beams. The writer's values, although somewhat conservative, can be considered accurate enough for design use.

In the "Specifications for the Design Of Light Gage Structural Members"[25] of the American Iron and Steel Institute, Eq. 54*a* was used to develop design requirements for nonbraced light-gage beams. For purposes of simplification, Eq. 54*a* can also be written in the following form:

$$f = \frac{74{,}000{,}000}{\left(\frac{l}{d}\right)^2}\left(\frac{r_y}{r_x}\right)^2 = \frac{74{,}000{,}000}{\left(\frac{l}{r_y}\right)^2}\left(\frac{d}{r_x}\right)^2 \quad \ldots\ldots\ldots\ldots (54b)$$

in which r_x and r_y are the radii of inertia of the section. It was found that for light-gage beams of economical dimensions the ratio d/r_x is almost constant and equal to 2.5 or slightly above that value. By substituting this value for d/r_x in Eq. 54*b*, the following, extremely simple formula for the buckling stress of light-gage beams of the type of Fig. 1 is obtained:

$$f = \frac{463{,}000{,}000}{\left(\frac{l}{r_y}\right)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (54c)$$

It is Eq. 54*c*, with a suitable factor of safety, which was used in the specifications of the American Iron and Steel Institute.

Although general indications concerning the range of applicability of these two formulas have been discussed herein, doubt may sometimes arise as to which of the two to use. In such a case it is a safe and somewhat conservative procedure to compute the buckling stresses by both approximate formulas (Eqs. 7 and 54) and to use the larger of the two values. The stress so obtained will always err on the conservative side, but by not more than about 30%. Indeed, both formulas give values which are always smaller than the exact stress (Eq. 51) since they neglect positive terms. The maximum error can

[25] "Specifications for the Design of Light Gage Structural Members," A.I.S.C., April, 1946.

easily be established for the worst case—that is, when both terms under the square root are equal. In that case either formula results in a stress which is $\sqrt{0.5}$, or 0.707, times the exact stress determined by Eq. 51. Any other ratio of these two terms, therefore, will cause the error of the larger of the two stresses to be less than 30%.

Both these investigations are strictly limited to I-shaped beams. Box beams or U-beams should not be designed by these formulas, since, for such beams, the equations give values that are often in error by hundreds of percentage points. Such members must be analyzed by substantially different methods. Current design specifications are rather vague in this respect and the author would have done well to indicate a clear limitation to I-beams in the title.

These remarks are in no way intended to detract from the value of this paper which, at long last, gives a usable formula for standard types of beams. On the contrary, the writer, despite his development of relatively simple rigorous equations, did not reduce them to their simplest possible approximate form. The author, on the other hand, although handicapped by his somewhat unnecessarily complex mathematics, developed a usable formula by persistent trial and error.

These comments, therefore, are merely intended to show the rigorous basis of the author's formulas, and to supplement his fundamental Eq. 7 by an equally simple one (Eq. 54*a*), which is valid for sections to which Mr. de Vries' formulas cannot be applied.

(In the numerical data presented in this discussion, the torsional constants were computed by the approximate formula:

$$K = (\tfrac{1}{3}) \Sigma w_n t^3_n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (55)$$

in which w_n and t_n are, respectively, the widths and thicknesses of the component rectangles of the section. Slide-rule accuracy was held to be sufficient for these merely illustrative figures.)

DAVID B. HALL,[26] ASSOC. M. ASCE.[26a]—Some of the beams analyzed in this paper have fairly high ratios of I_y to I_x. Since the formulas of the paper are all derived on the assumption that this ratio is negligibly small, the question arises as to the amount of error introduced. It turns out, however, that, in practice, these beams can never buckle within the elastic range. Hence an increase in stability which, in an elastic structure could permit a considerable increase in the buckling load, would increase the load only slightly in this case. As an illustration, the 14WF87 beam, with a 14.5-in. flange may be considered. Referring to Table 2, it is observed that the lowest value of l/a for which the critical stress f was small enough to be recorded was 4. Since the value of a is 104 in., this would give a length of 4×104, or 416 in., which is approximately thirty times the depth of the beam. To prevent excessive deflection, the maximum length permitted in practically any type of structure would be considerably less than this. It may thus be concluded that questions

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[26a] Received October 25, 1946.

of refinement of the theoretical buckling equations in this region are subordinate to the broader problem of plastic buckling, which the writer is not prepared to discuss.

The academic problem of such a beam, long enough to buckle in the elastic range (about 700 in. for a 14 × 14.5 beam) may still be of interest. The modified buckling load depends on the ratio I_y/I_z and also upon a relation between the torsional stiffness (including torsion bending) and I_z, but for a long I-beam the latter relation is of negligible importance. For any type of loading applied along the centroid of the beam the approximate correct buckling load can then be found by multiplying the buckling load that was previously found by the methods of the paper, by a factor,

$$F = \frac{1}{\sqrt{1 - \frac{I_y}{I_z}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (56)$$

For the 14WF87 beam $F = 1.25$. Eq. 56 is easily obtained by considerations of energy. Assuming any section rotated through an angle β, and acted on by a moment M about the x-axis, the moments about the two principal axes of the section are $M \cos \beta$ and $M \sin \beta$, and the components of the curvature in the (yz)-plane are $\frac{M}{E I_z} \cos^2 \beta$ and $\frac{M}{E I_y} \sin^2 \beta$. Since $\cos^2 \beta = 1 - \sin^2 \beta$,

$$\text{The total curvature of an element} = \frac{M}{E I_z} + \frac{M}{E I_y} (1 - I_y/I_z) \sin^2 \beta \ldots (57)$$

The first term in Eq. 57 represents the curvature in the stable state, and the second represents the additional curvature in the buckled state. In this latter term $\sin^2 \beta$ may be replaced by β^2, and it is found that the expression $\frac{1 - I_y/I_z}{E I_y}$ takes the place of $\frac{1}{E I_y}$ in expressions for both internal and external energy, and ultimately comes out in the form of Eq. 56.

A second question concerns the selection of loading and end support conditions. Several conditions have been treated in the paper. One interesting and very simple case (that of a simple beam acted on by pure end couples) was omitted.

A common type of bridge structure consists of a pair of main carrying beams or girders braced together by floor beams. The weight of the floor and moving traffic is delivered to the main beams at the floor-beam connections. Thus a single panel of such a beam corresponds very closely to the ideal beam. The condition of simply supported ends follows from the freedom of alternate panels to buckle in opposite directions, although it is modified somewhat by the restraint offered by the floor beams and bracing.

Beams in which principal loads are applied between supporting points will ordinarily present a more complicated problem than that treated in the paper, since structures of this kind (for example, a crane runway beam) would almost certainly be subject to lateral as well as vertical loading.

The case of a simple beam acted on by end couples is easier to treat than any of those considered in the paper. The coefficient k in Eq. 4 can be computed directly instead of by trial.[27] A few typical values are:

l/a	k
1	2.92
2	3.41
10	8.2

Comparison with values in Table 1 indicates that this condition is about eight tenths as severe as the case of a beam uniformly loaded along the top flange, when l/a is 1 and that it is practically identical to it when l/a is 10. It would appear logical to choose this as the standard loading condition, treating the other cases as special.

In the paper, the case of a long girder picked up at the ends during erection was cited as an example of the case of a simply supported beam loaded along the center line. Such a beam, supported by ordinary slings, is prevented from rotating about a point in the vicinity of the top flange only by its own weight, and presents an entirely different problem. For equilibrium in the slightly buckled state, the product of the height from center of load to the eye of the sling by the end rotation β must be equal to the average lateral deflection. An equilibrium solution, however, would be difficult except in the case where the torsional rigidity of the beam is assumed infinite. In this case the angle β is constant throughout the length of the beam. The lateral load is $w\beta$. The average lateral deflection, obtained by integrating the deflection curve, is found to be $\frac{w l^4 \beta}{120 E I_y}$. The height from load to the eye of sling may be conservatively taken as $d/2$, making the offset $\frac{\beta d}{2}$. Then:

$$\frac{w l^4 \beta}{120 E I_y} = \frac{\beta d}{2} \quad \ldots\ldots (58a)$$

and

$$\frac{w l^4}{E I_y d} = 60 \quad \ldots\ldots (58b)$$

By properly combining Eqs. 58*b*, 32, and 4, there follows the relation,

$$k = 3.75 \quad \ldots\ldots (58c)$$

which represents a limiting value approached by k as l/a increases indefinitely. This value of k contrasts strongly with the indefinitely increasing value of k in Table 1, for cases in which end rotation is prevented.

An approximate solution can be obtained by the energy method. An equation very similar to Eq. 28 may be derived, replacing the term from Eq. 25 (for lowering a load distributed along a flange) by the expression,

$$V_2 = -\frac{\beta^2_{z=0.5l}}{2} w l \frac{d}{2} \quad \ldots\ldots (59)$$

[27] "Theory of Elastic Stability," by S. Timoshenko, McGraw-Hill Book Co., Inc., New York and London, 1936, p. 261, Eq. 159.

to represent the work done in raising the entire load when the ends rotate. The new equation becomes:

$$\frac{w^2}{4\,E\,I_y}\int_0^{l/2}\beta^2\left(\frac{l^2}{4}-z^2\right)dz-\frac{w\,l\,d}{4}\beta^2_{z=0.5l}-\frac{E\,I_y}{4}\frac{l^2}{a^2}\frac{d^2}{l^2}\int_0^{l/2}\left(\frac{d\beta}{dz}\right)^2dz$$
$$-\frac{E\,I_y}{4}\,d^2\int_0^{l/2}\left(\frac{d^2\beta}{dz^2}\right)^2dz=0\ldots\ldots\ldots\ldots\ldots\ldots(60)$$

A reasonably satisfactory representation of β as a function of z should be:

$$\beta=a_0+a_1\cos\frac{\pi\,z}{l}=a_0\left(1+m\cos\frac{\pi\,z}{l}\right)\ldots\ldots\ldots\ldots(61)$$

Substitution of Eq. 61 and its derivatives in Eq. 60 yields:

$$\frac{w^2\,l^5}{4\,E\,I_y}\left[\frac{1}{60}+m\left(-\frac{4}{\pi^3}+\frac{48}{\pi^5}\right)+m^2\left(\frac{1}{120}+\frac{3}{8\,\pi^4}\right)\right]$$
$$-\frac{w\,l\,d}{4}-\frac{E\,I_y}{4}\frac{l^2}{a^2}\frac{d^2}{l^2}\frac{m^2\,\pi^2}{4\,l}-\frac{E\,I_y}{4}\,d^2\frac{m^2\,\pi^4}{4\,l^3}=0\ldots\ldots\ldots(62)$$

Let:

$$\frac{w\,l^4}{E\,I_y\,d}=X\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(63)$$

Then Eq. 62 reduces to:

$$X^2\left[\frac{1}{60}+m\left(-\frac{4}{\pi^3}+\frac{48}{\pi^5}\right)+m^2\left(\frac{1}{120}+\frac{3}{8\,\pi^4}\right)\right]$$
$$-X-\frac{\pi^2\,m^2}{4}\left(\frac{l^2}{a^2}+\pi^2\right)=0\ldots\ldots\ldots\ldots\ldots(64)$$

Eq. 64 has been solved approximately for a few typical values of l/a, with the following results:

l/a	X	k
2	17.85	1.12
8	27.2	1.70
∞	60	3.75

The last line, it will be noted, checks with Eqs. 58*b* and 58*c*.

This analysis shows that the handling of a long flexible girder during erection is likely to require special consideration, and also indicates that one possible way to strengthen such a girder is by devising a special lifting rig with a high center of rotation.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

DRAWDOWN TEST TO DETERMINE EFFECTIVE RADIUS OF ARTESIAN WELL

Discussion

BY CARL ROHWER

CARL ROHWER,[12] M. ASCE.[12a]—The serious depletion of ground-water supplies in many areas during World War II has focused attention on the problems of ground-water hydrology. In this connection the investigations of the engineers of the Water Resources Branch of the U. S. Geological Survey are adding important information regarding the characteristics of wells and the capacity of ground-water formations. The analysis of drawdown tests of artesian wells by the author is a valuable contribution to this subject.

The writer is in agreement with the objectives of the author's investigations but he is of the opinion that the analysis of the problem would have been simplified if some of the factors that have only a slight effect on the results had been ignored. Under most conditions met with in the field of engineering the compressibility of water can be ignored. The coefficient is approximately 4×10^{-6} per pound pressure at ordinary temperatures and pressures. A reduction in pressure of 10 lb per sq in. would increase the volume of 1 cu ft of water by only 0.00004 cu ft, a difference of 1 in 25,000. In view of the large unavoidable errors involved in other measurements it seems that this factor could well be neglected. The same may be stated of the compression of the aquifer. As indicated by the author (see heading, "Application of Theory to a Simple Drawdown Test"), the combined effect of compressibility of the water-bearing formation is only five times the actual compressibility of water. Consequently, the combination of these two factors would produce a change of only 1 in 5,000 for a drop in pressure of 10 lb (approximately 23 ft). If the change in pressure were increased to 100 ft the effect produced by the compressibility of the water and aquifer would not be significant.

NOTE.—This paper by C. E. Jacob was published in May, 1946, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1946, by N. S. Boulton.

[12] Senior Irrig. Engr., Div. of Irrig., Soil Conservation Service, U. S. Dept. of Agriculture, Fort Collins, Colo.

[12a] Received October 30, 1946.

In reference to the tests on a shallow well at Meadville, Pa., the author states in the sentences following Eq. 18, that:

> "The foregoing calculations indicate that $B\,Q$ in this case was about 19.5 ft after 24 hours of continuous pumping. The observed drawdown in the pumping well at that time was 48.0 ft, leaving 28.5 ft for the well loss."

Such a large well loss seems unusual for an inflow of 1,350 gal per min through 15 ft of 18-in. screen unless the screen were badly encrusted or improperly perforated. Immediate steps should be taken to improve the performance of the screen in this well.

In the solution of problems involving many variables of which only a few can be determined by direct measurement, the use of multiple equations provides a method of determining the unknowns. However, there are difficulties inherent in this method which may lead to contradictory or inconsistent results. As shown by the author in reference to the determination of C (see heading, "Data from Multiple-Step Drawdown Test"), there is considerable scattering in the values obtained for the "specific incremental drawdown," Fig. 7(*d*). No oubt, this is due in part to the inaccuracies in the drawdown readings. If this method were used on problems in which all readings could be made accurately, consistent results should be expected. Since this is not generally true, the multiple-equation method results in solutions in which the final answers may have errors greatly in excess of the observed data. The writer is not aware of the mathematical basis for this assumption, but he observed the same effect when attempting to use a similar method to determine the values of the factors involved in the seepage from canals. The conclusion was reached that, in the elimination of variables from the series of equations by subtraction, the variables eliminated were forced to conform exactly to the law; and, as a result, all the discrepancies accumulated and finally appeared in the solution of the unknown. A solution based on another pair of equations may, therefore, yield a result widely different from the first one.

Since the author has had the opportunity to observe how the solutions vary when he uses different equations it would be of interest to study the mathematical principles causing the variations. No doubt rules could be formulated which would make it possible to obtain more consistent results from the observed data. Such an analysis would be useful in the solution of problems in other fields of engineering.

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DISCUSSIONS

SPACE RESECTION PROBLEMS IN PHOTOGRAMMETRY

Discussion

BY EARL CHURCH

EARL CHURCH,[5] ASSOC. M. ASCE.[5a]—Although this paper is entitled "Space Resection Problems in Photogrammetry," in reality it gives a solution of the space orientation problem (finding certain quantities or elements which serve to specify uniquely the orientation of the aerial photograph in space) as well as of the space resection problem (finding the survey space coordinates of the exposure station). These two problems are often treated together, as has been done by Professor Underwood.

In this paper the space resection problem has been approached by the often tried but seldom used pyramid method—determining the desired position of the exposure station by the lengths of the pyramid edges. The particular solution of this problem is very interesting, especially the use of Eq. 13 for finding the ratio u representing $d\rho_{A'}/d\rho_A$.

The method has two detractive features: (1) The solutions of the triangles forming the pyramid faces are tedious, inasmuch as they must always be made twice and often three times; and (2) the completion of this phase of the problem gives only the corrected lengths of the pyramid edges, still leaving the desired survey coordinates of the exposure station undetermined.

The part of the computation for finding these coordinates, simple as it is in principle, involving only the solution of the three simultaneous Eqs. 18, is, of course, troublesome in practice. Professor Underwood proposes a solution by approximations involving two preliminary computations, one consisting of the three-point problem in a plane and the other of the simple scale problem of photogrammetry. He then solves Eqs. 18, by the use of k as explained in the text following Eqs. 22 and cited in the "Summary." Possibly the straightforward algebraic solution of Eqs. 18 (of three spheres) with no preliminary computations, might even be more time saving than this method of approximations.

NOTE.—This paper by P. H. Underwood was published in September, 1946, *Proceedings*.

[5] Prof. of Photogrammetry, College of Applied Science, Syracuse Univ., Syracuse, N. Y.

[5a] Received November 1, 1946.

In either case, Professor Underwood's solution of the space resection problem is interesting; and, although perhaps somewhat longer than the usual space-analytics method, it is certainly not long enough to be impracticable.

Then space orientation is considered. The solution shows an interesting method of determining both the intersection of the photo plane with the vertical line through the exposure station and the survey position of the intersection of the camera axis with the ground plane. The former obviously determines at once the tilt and the swing, and the latter determines indirectly the azimuth of the principal plane of the photograph. The graphical determination of these two points, despite the large-scale drawings (Figs. 12 and 13), leaves something to be desired. What appear to be "triangles of error" in these graphs, as Professor Underwood states, must not be regarded as triangles of error, as far as errors in either control surveying or photographic measuring are concerned. They are triangles of error in either the analytical or the graphical phases of the computation itself.

On the whole, Professor Underwood's solution of the space orientation problem is of considerable interest because of the manner in which it attacks essential features of the problem. However, instructive as it is, the method compares unfavorably with several equally precise space-analytics methods in both simplicity and time required for the computations.

Professor Underwood's illustrative computation, although entirely correct, scarcely does justice to his method. The initial values for the desired quantities in his solution, obtained by methods of approximation, are much nearer to the correct values than would ordinarily be available in actual practice. This result might cause suspicion regarding convergence of the solution, should less accurate initial values be used. However, in the writer's experimental work with Professor Underwood's method, the solutions have been found to converge very satisfactorily under far less favorable conditions than those shown in the paper.

It must always be remembered that only by consideration of all the various solutions advanced can the most practical methods be evolved for handling these complex problems in practice. Professor Underwood's paper, therefore, deserves thorough reading and careful consideration by all photogrammetrists.

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DISCUSSIONS

TECHNICAL SURVEY—BROOKLYN BRIDGE AFTER SIXTY YEARS A SYMPOSIUM

Discussion

BY NOMER GRAY

NOMER GRAY,[31] M. ASCE.[31a]—In commenting on the statement in the writer's paper that "passing trains produced a deflection at the center of the main span in excess of 1 ft," Mr. Thomson states that "the total possible deflection is 18 ft, 9 ft above and 9 ft below the mean level." Even as the most extreme hypothetical deflection at midspan that one could rationalize, 18 ft is much too large for Brooklyn Bridge. The maximum range of main-span deflection is about 6 ft—3 ft above and 3 ft below a mean level. This estimate is based on a temperature range of from 120° F to − 10° F, and a live load such as prevailed in the first decade of the twentieth century, when the bridge was subjected to the heaviest traffic. Elevated railway service was discontinued on March 5, 1944. With the present live load, consisting of trolley cars and passenger automobiles, the corresponding maximum range of deflection is not likely to exceed 4 ft.

The writer takes this opportunity of expressing the appreciation of the authors of the four papers comprising the Symposium for the contributions made by several discussers.

NOTE.—This Symposium was published in January, 1946, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1946, by Isidore Delson, and T. Kennard Thomson; May, 1946, by Theodore Belzner; and June, 1946, by C. H. Gronquist, and Blair Birdsall.

[31] With Ammann and Whitney, Cons. Engrs., New York, N. Y.

[31a] Received November 7, 1946.

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DISCUSSIONS

TOPOGRAPHIC SURVEYS

PROGRESS REPORT OF THE COMMITTEE OF THE SURVEYING AND MAPPING DIVISION ON TOPOGRAPHIC SURVEYS

Discussion

BY PAUL VAN BUSKIRK

PAUL VAN BUSKIRK,[6] M. ASCE.[6a]—This progress report serves to bring into relief some of the controversial features relating to topography.

Few engineers will rely on volume quantities computed from a contour map, although it can be demonstrated that the data obtainable from a properly made contour map are superior to those on the usual job of cross sectioning, where the "inside" breaks are not taken. Particularly on preliminary estimates much money is wasted on cross sectioning that could be saved were a little more skill exercised in the preparation of the topographic map.

This raises the old argument as to the best method of filling in detail. Experience in the Detroit (Mich.) district seems to show that the proper scale for its purposes is 1 in. equals 100 ft, using a 2-ft contour interval. The plane-table method has been discarded, although all topography is plotted in the field, and contours are drawn as located. Horizontal control is from the section lines and the quarter-section lines, all of which have been rerun, since 1936, to second-order standards.

The engineers of the Wayne County (Michigan) Road Commission use an interesting variation of the combined method. They plot their control on a plane-table sheet at 1 in. equals 100 ft, field plot their transit and stadia data, using a full-circle 10-in. protractor, reading counterclockwise, having a graduated scale on the crossbar. This is pivoted at the center with a needle through the station, another needle 5 in. north of the station serving as a north point. The azimuth to be plotted is brought to the north point, and the distance pointed along the scale. These sheets are inked, reduced photographically to a 200-ft scale, and traced as land sections.

NOTE.—This report was published in April, 1946, *Proceedings*. Discussion on this report has appeared in *Proceedings*, as follows: October, 1946, by Roger E. Amidon, Daniel Kennedy, and Ralph P. Black; and November, 1946, by Benjamin E. Beavin.

[6] Location Engr., Huron-Clinton Metropolitan Authority, Detroit, Mich.

[6a] Received October 31, 1946.

AMERICAN SOCIETY OF CIVIL ENGINEERS

COMING MEETINGS

BOARD OF DIRECTION MEETING

January 13-14, 1947:

A Quarterly Meeting will be held in New York, N. Y.

ANNUAL MEETING

HOTEL COMMODORE, NEW YORK, N. Y.

January 15-18, 1947

The Reading Room of the Society is open from 9:00 A.M. to 5:00 P.M. every day, except Saturdays when it is closed at 12:00 noon. It is closed all day on Sundays and holidays.

Members, particularly those from out of town, are cordially invited to use this room on their visits to New York, to have their mail addressed there, and to utilize it as a place for meeting others. There is an ample file of current periodicals, the latest technical books, and the room is well supplied with writing tables.